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LEADING CONTENTS

	PAGE
The Terminology of Prestressed Concrete	159
Design of Statically-Indeterminate Structures by the "Plastic-hinge" Method *By R. Gartner*	161
Office Building in London with "Flat-plate" Floors and Horseshoe-shaped Staircase	175
A Precast Cooling Tower	181
Precast Shell Roofs *By A. N. S. Kulasinghe*	183
Book Reviews	186
Graphical Determination of the Grading of Combined Aggregates	187
Cements for Massive Concrete Structures	191
Automatic Assembly of Reinforcement	193

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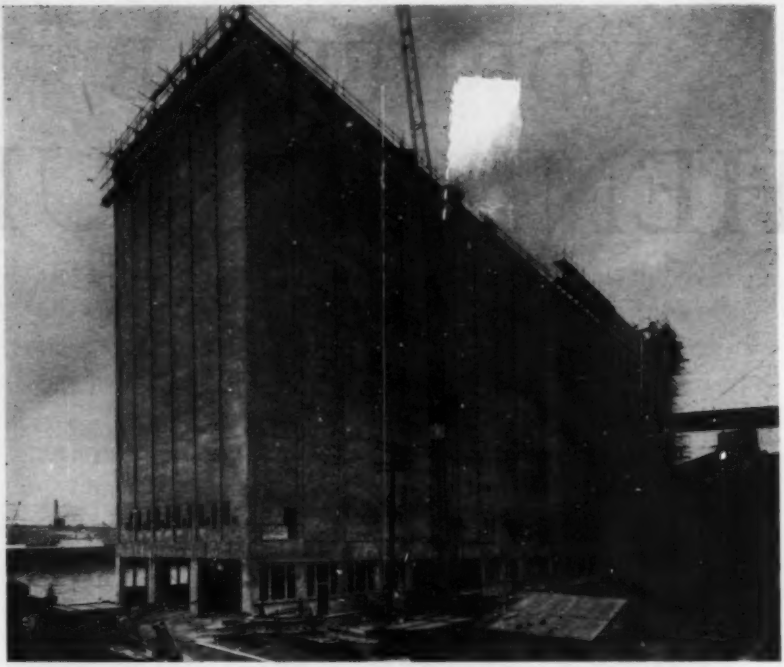

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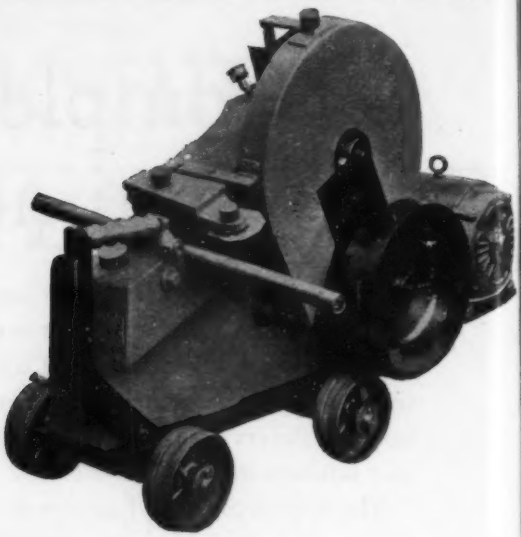

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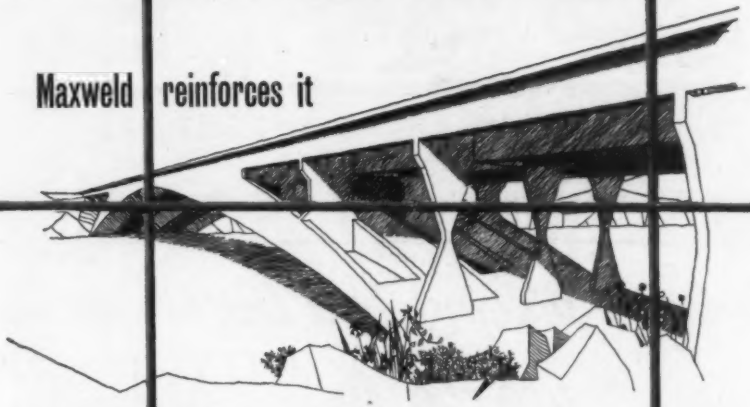

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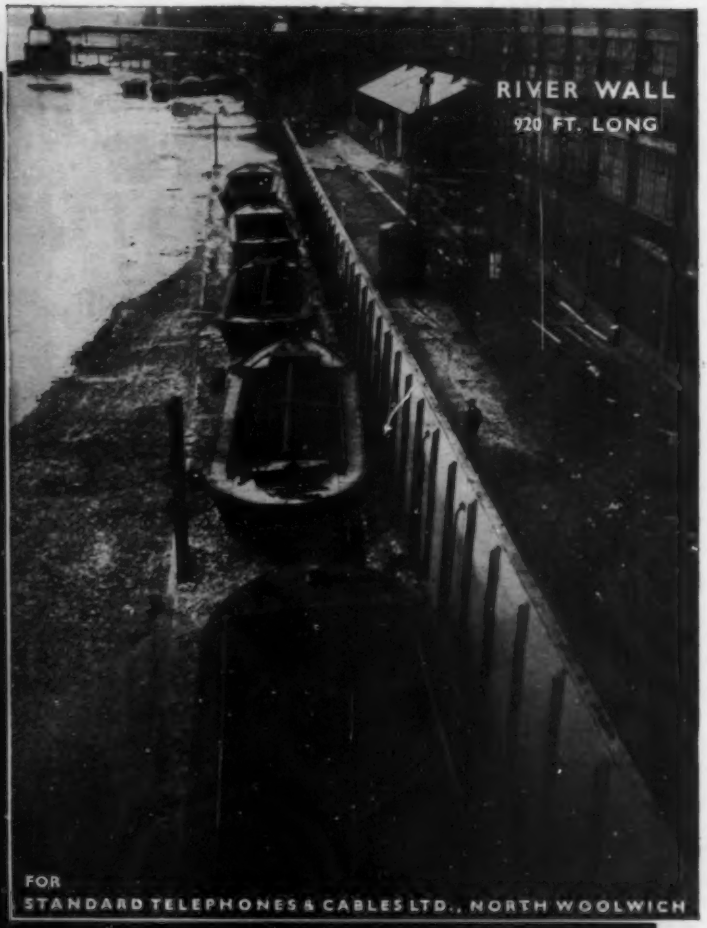

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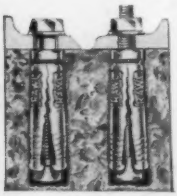

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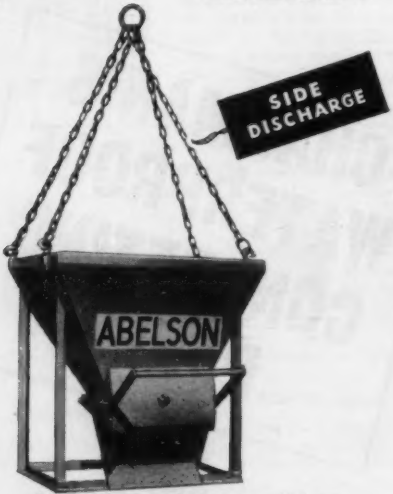

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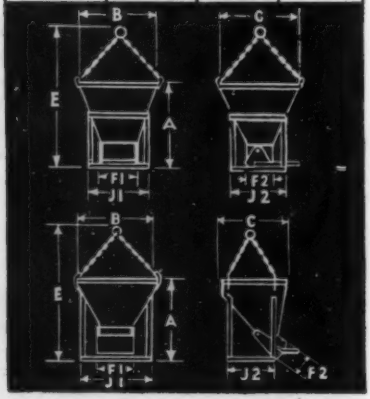

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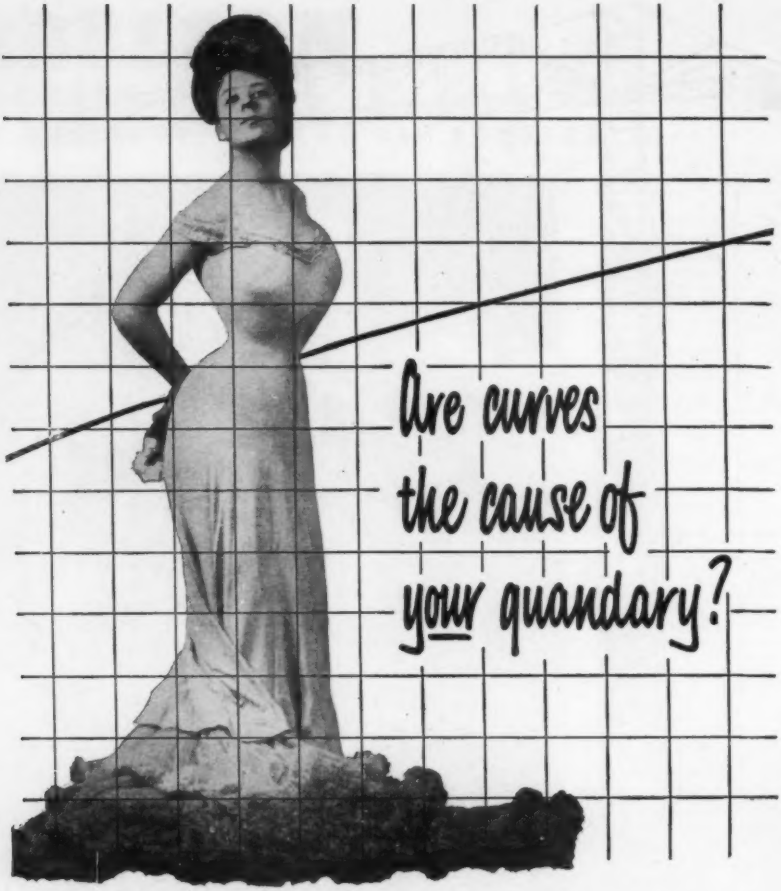

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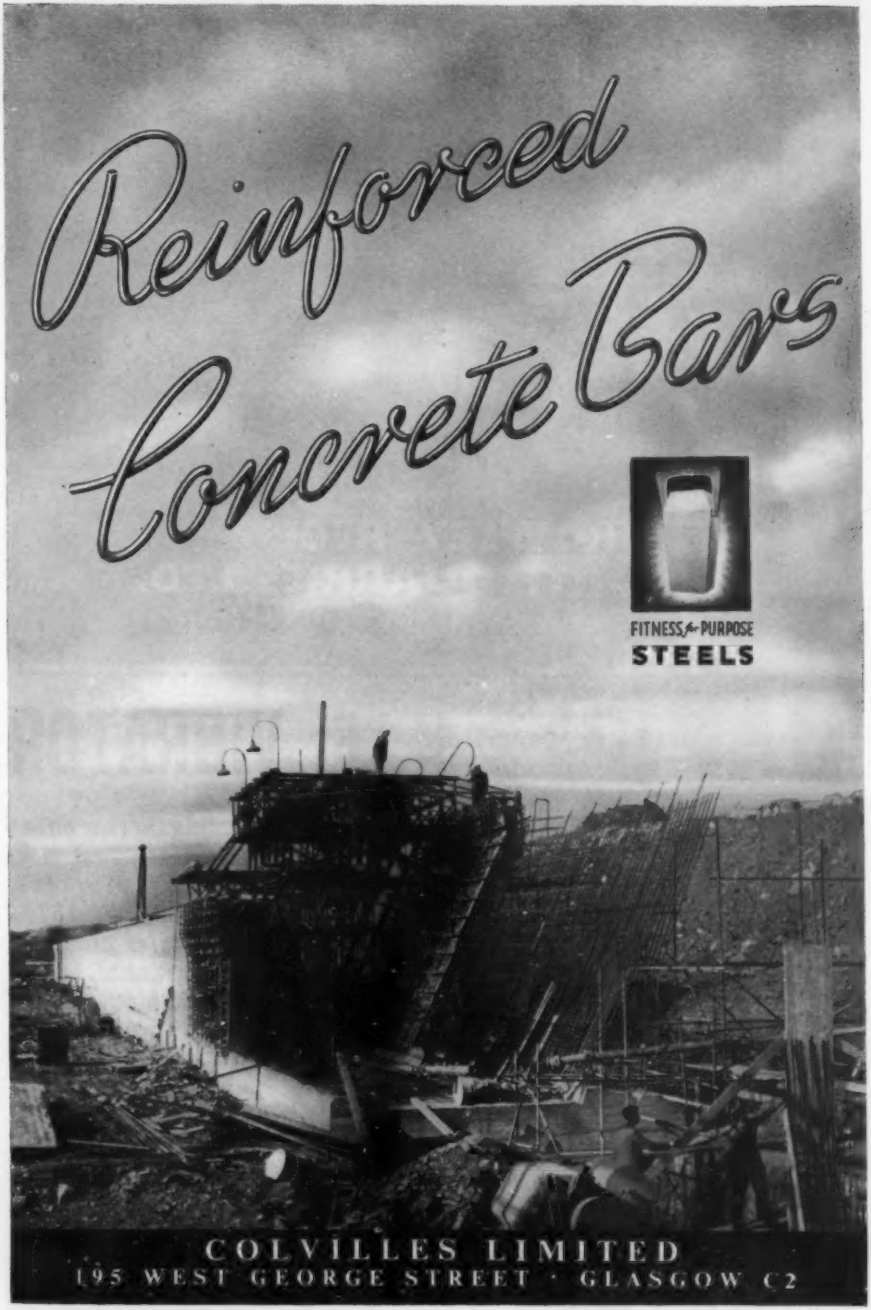
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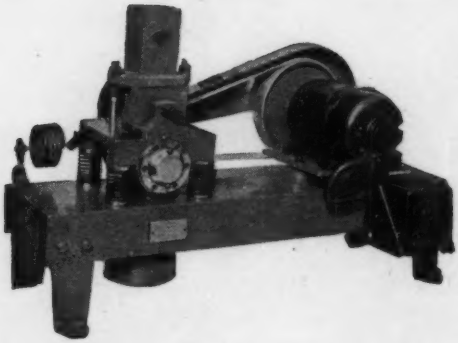

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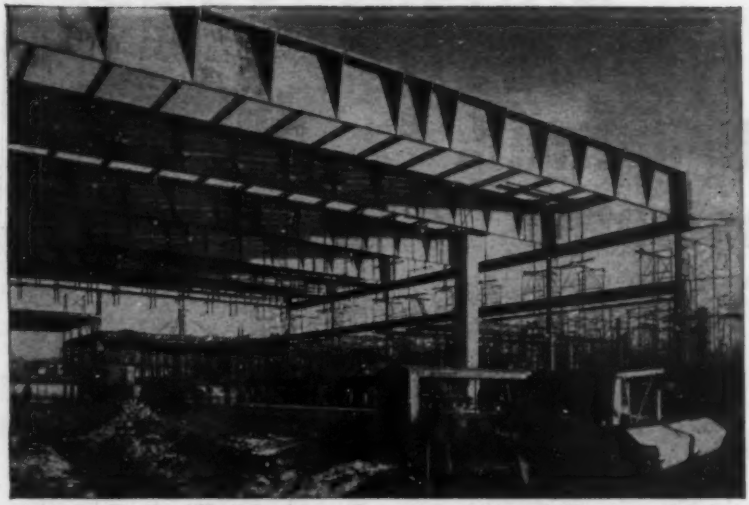

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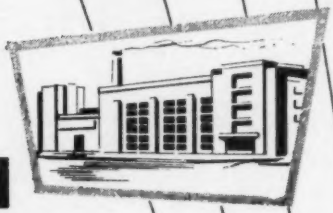

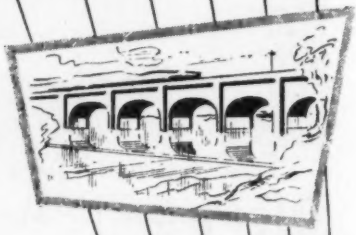

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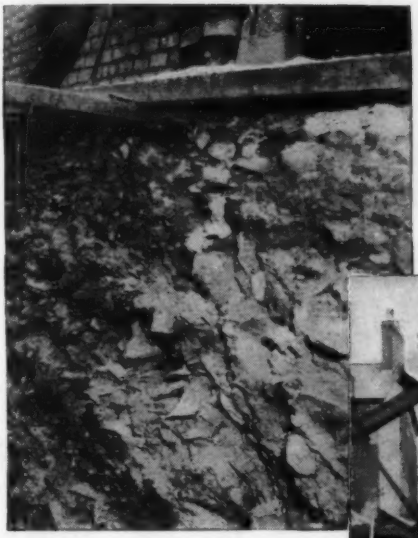

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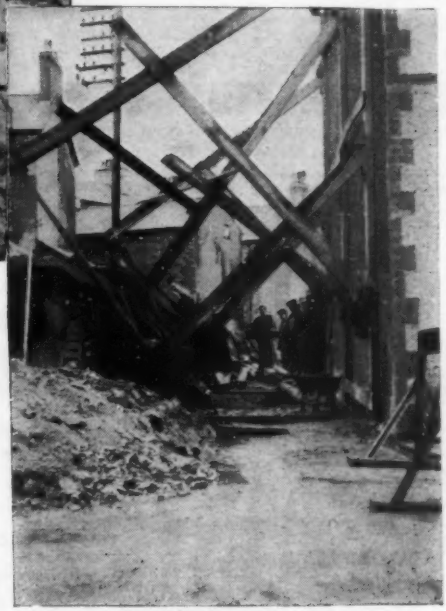

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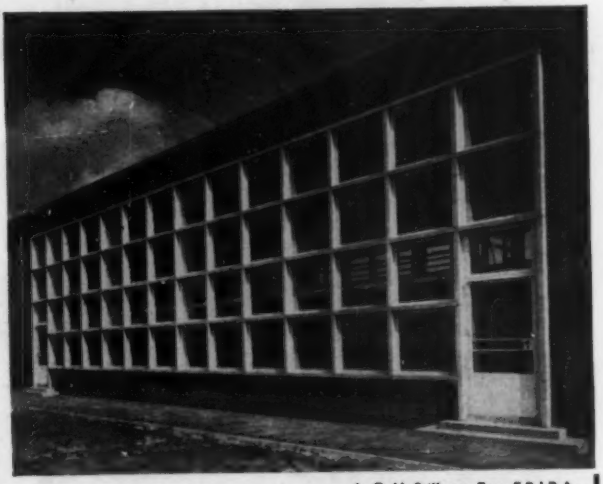

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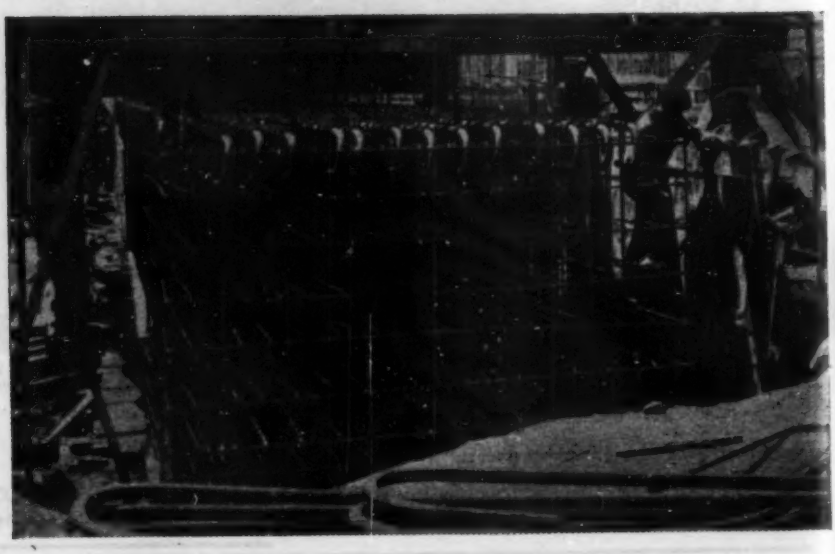

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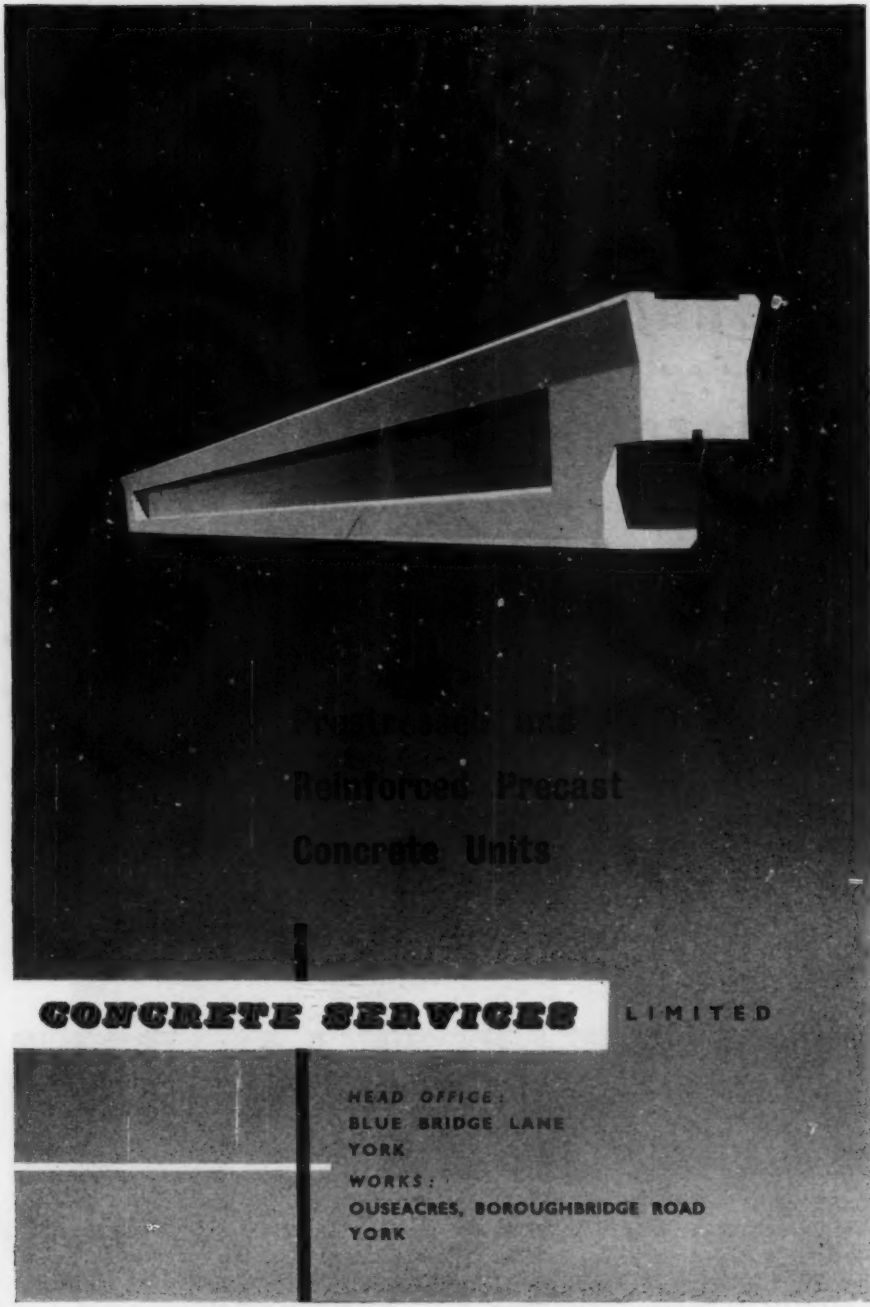
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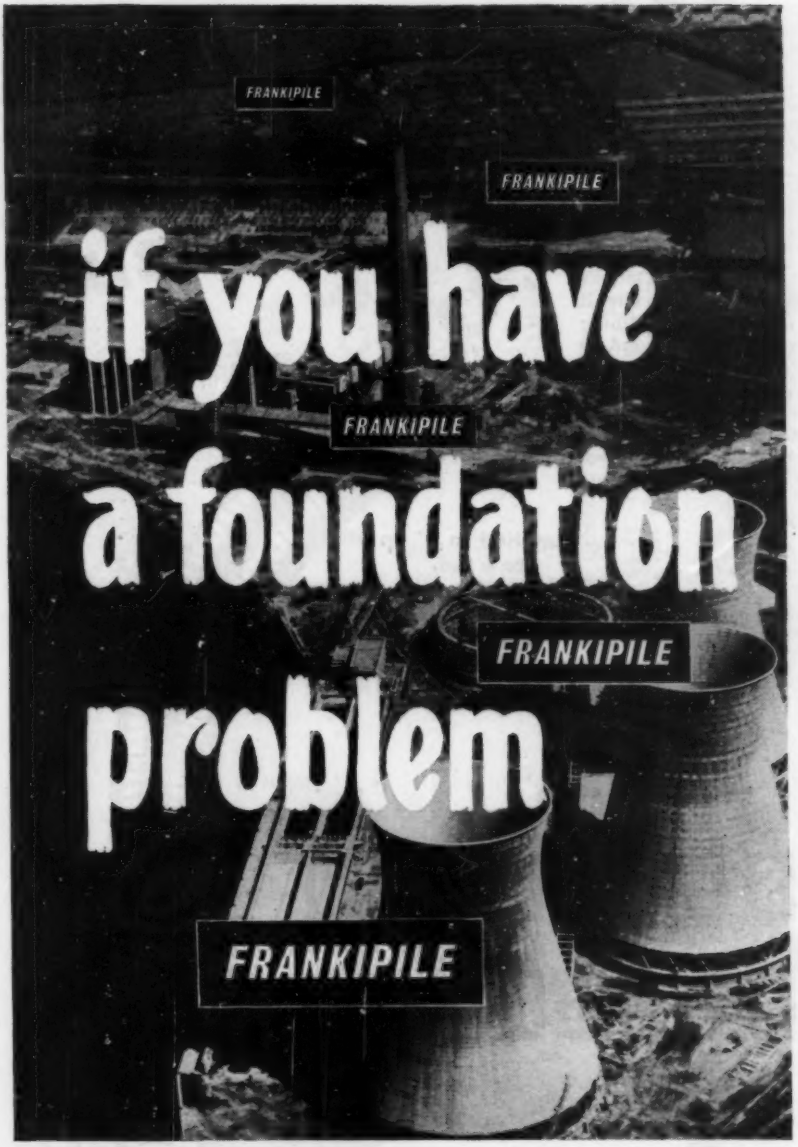

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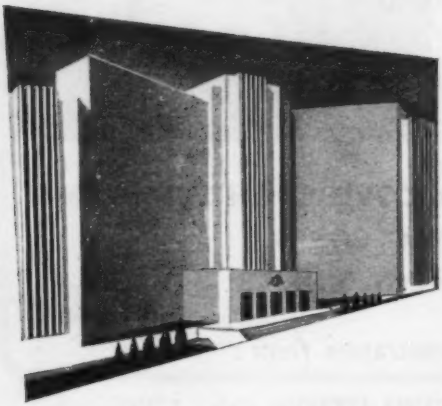

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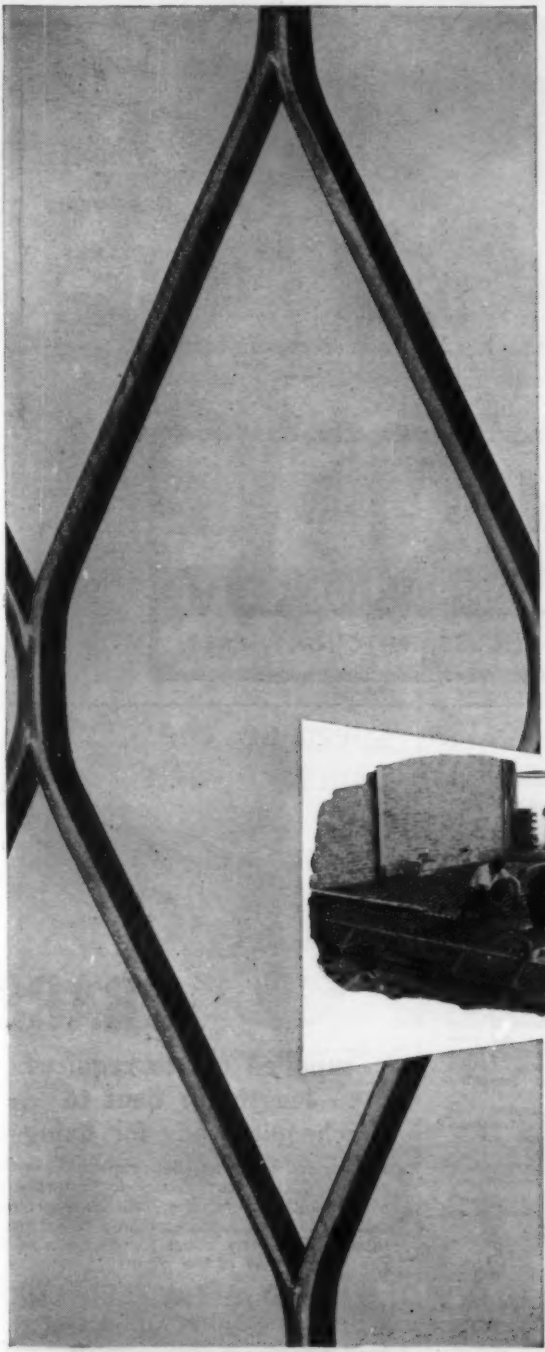

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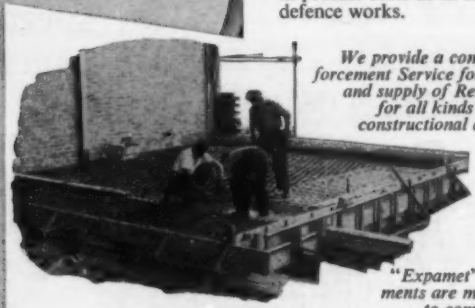

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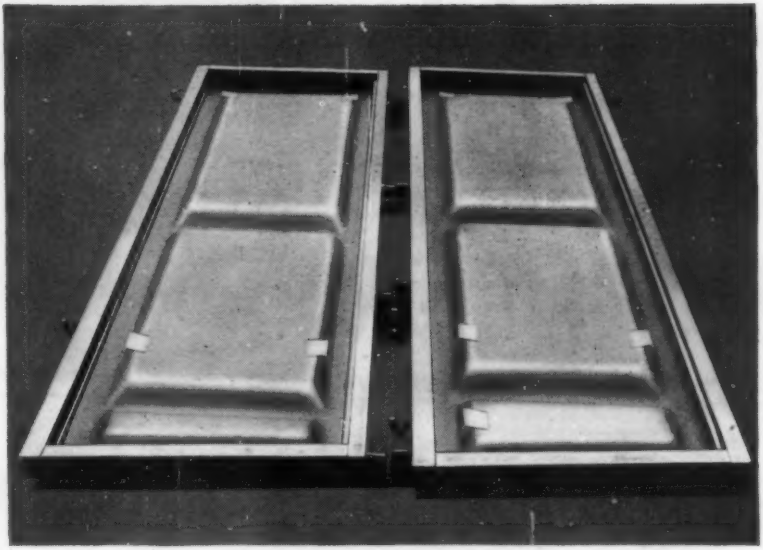

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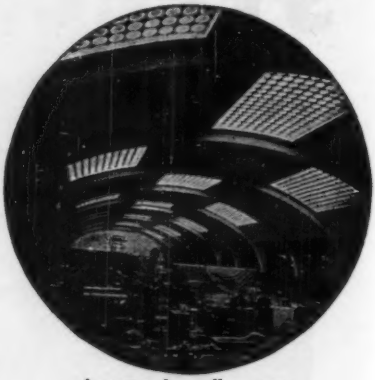

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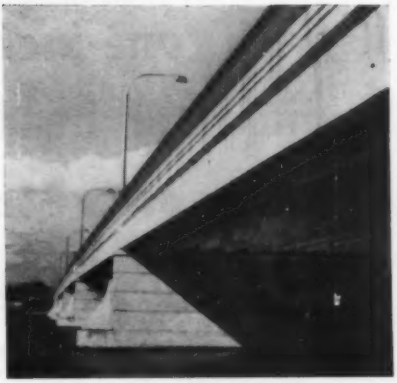

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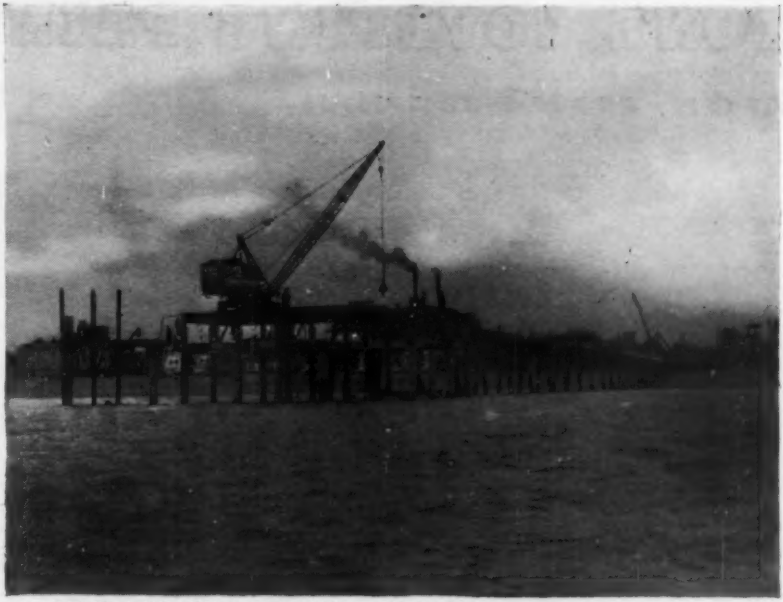

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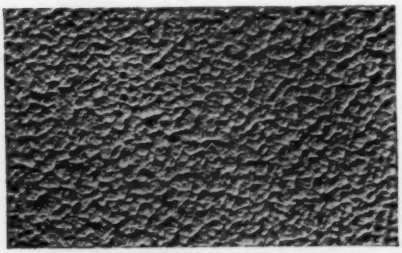

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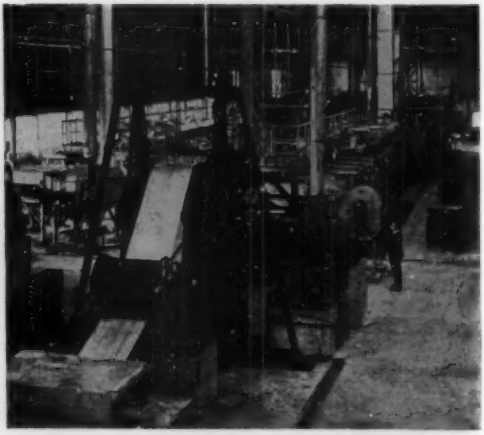

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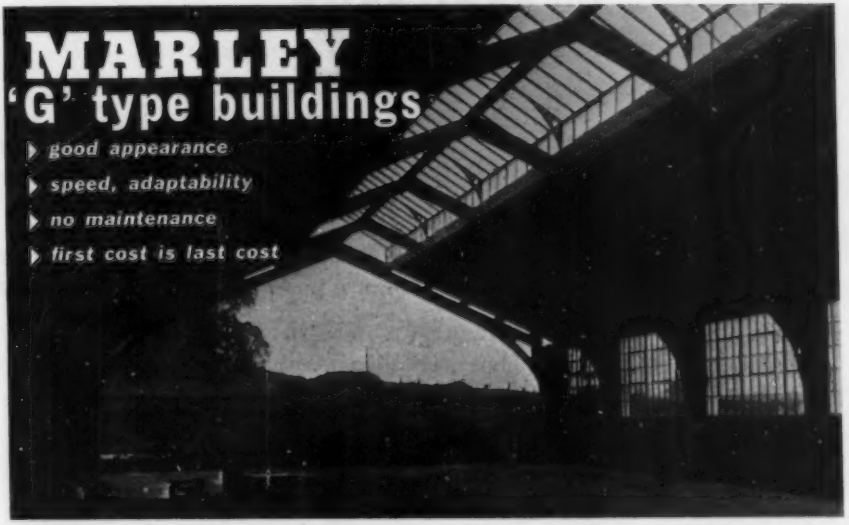

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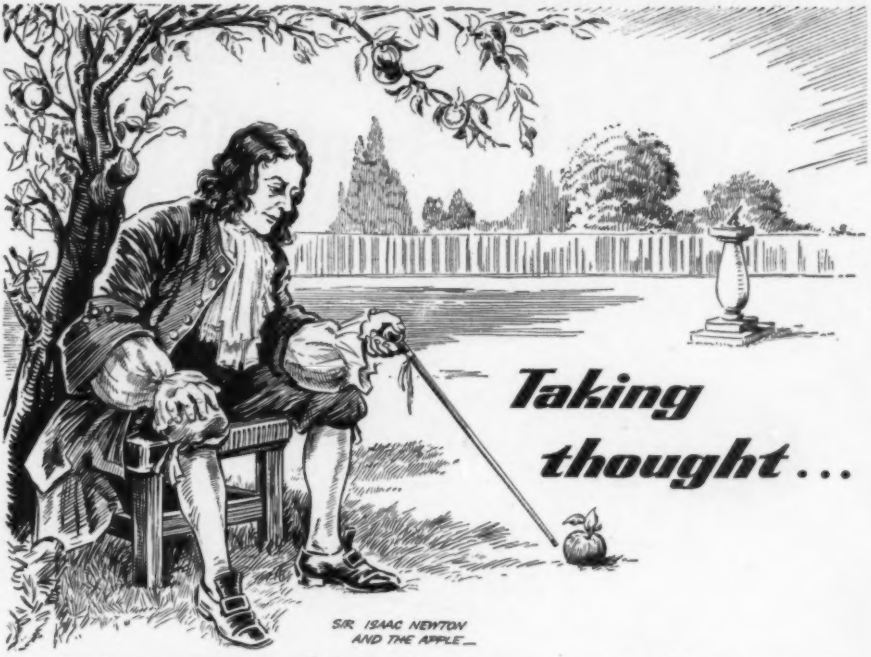

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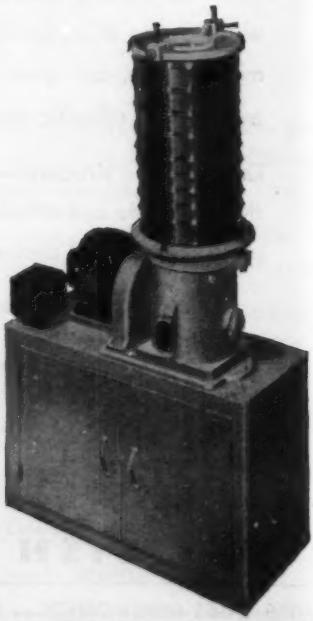

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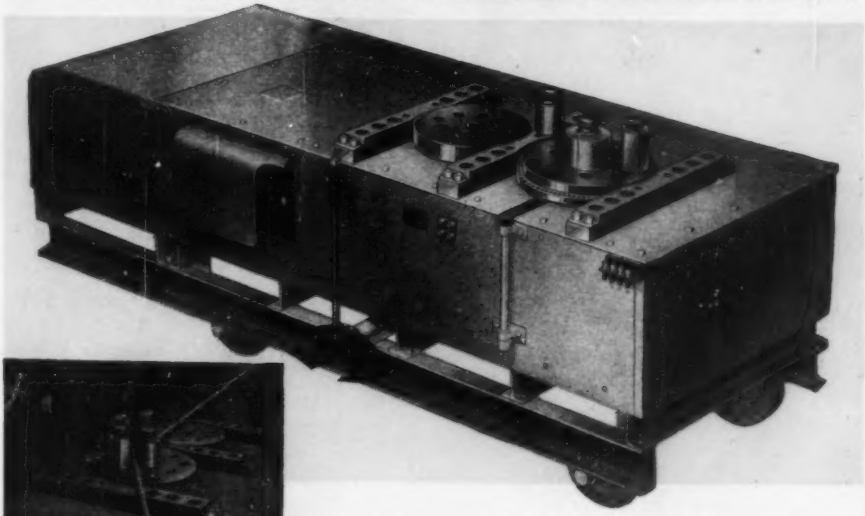

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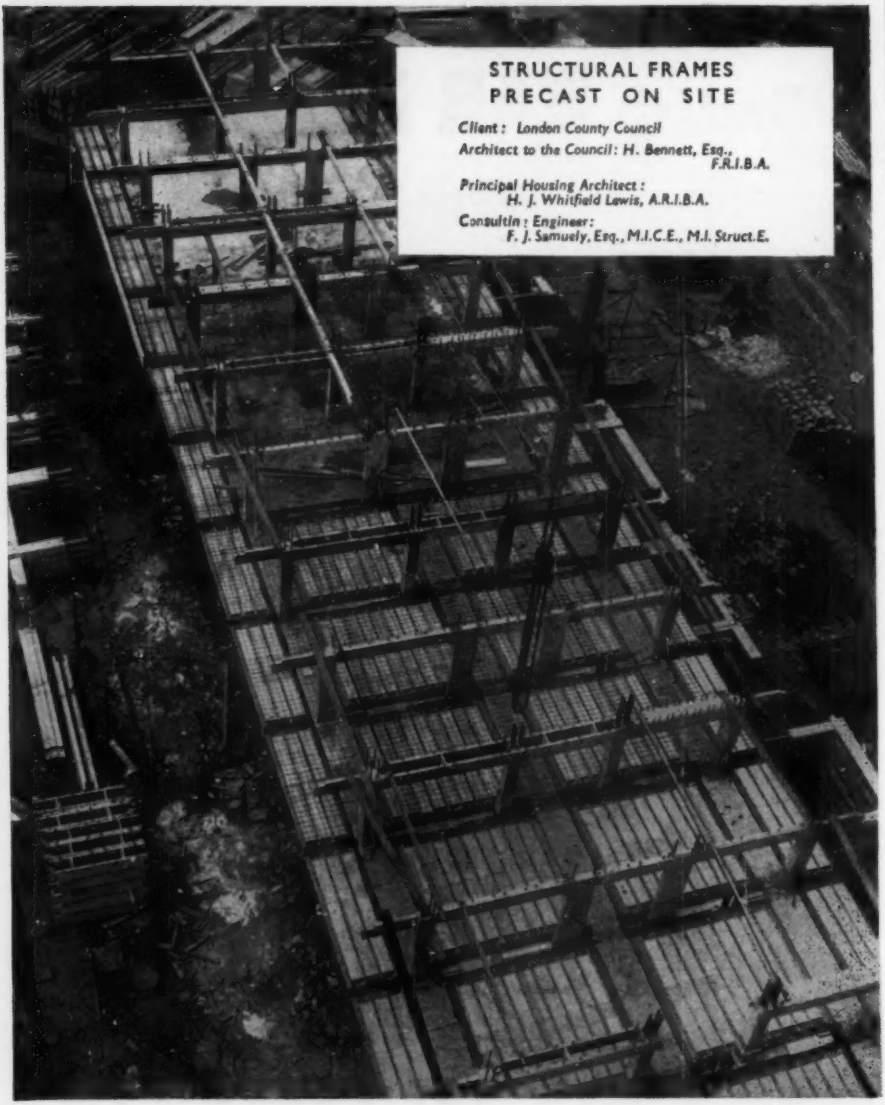
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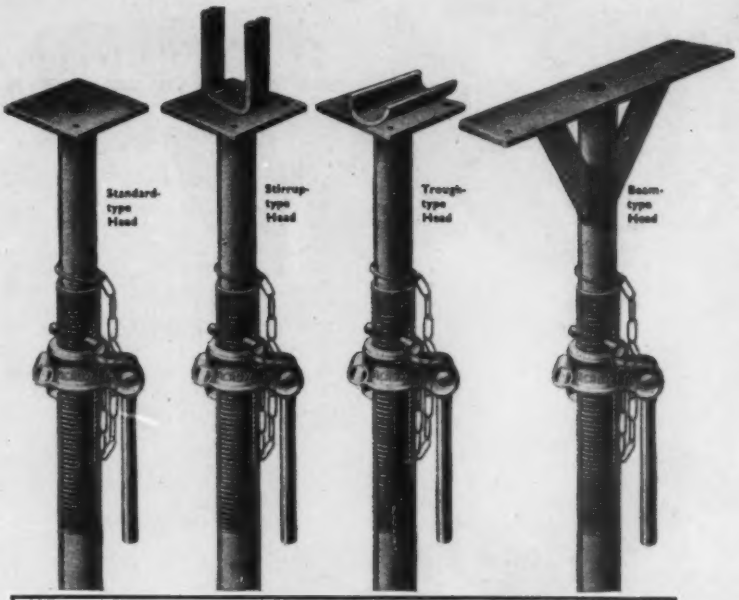

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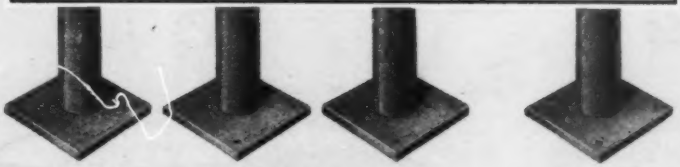

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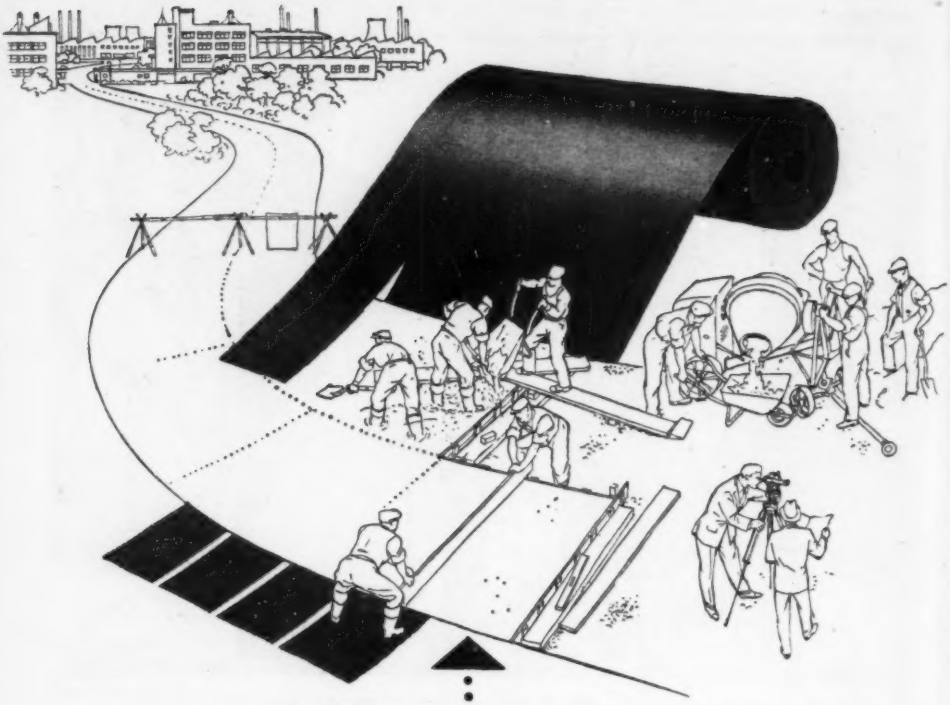

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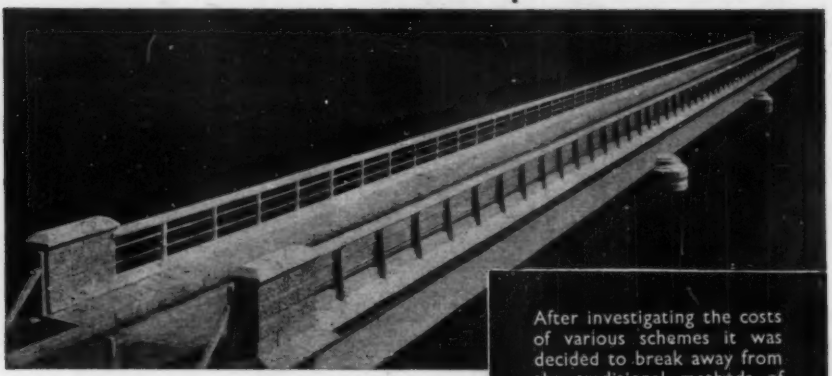

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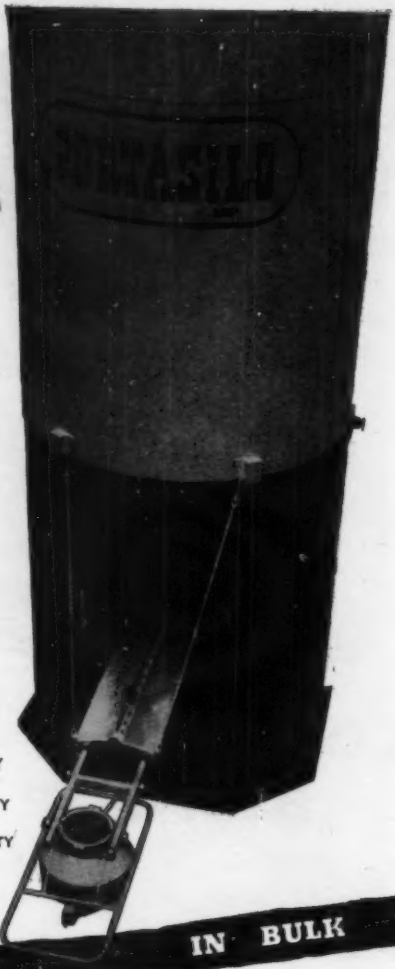

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Volume LII, No. 5. LONDON, MAY, 1957.

EDITORIAL NOTES

The Terminology of Prestressed Concrete.

PRESTRESSED concrete has been bedevilled by the terminology that has grown up around it in a shockingly haphazard manner. The term prestressed is itself meaningless. There are various kinds of stress to which a material may be subjected, and if we say merely that it is prestressed we do not indicate whether it has been stretched, or squeezed, or twisted, or what has been done to it. Compression is a better translation of the French word *contraint* from which prestressed (*précontraint*) is derived, and has the advantage of indicating exactly what has happened to the concrete. The careless use of the words stress and prestress fogs much of the writing and talk on the subject, as is shown by the following letter received from a correspondent:

> "I was very pleased to see the Editorial Note in your February number drawing attention to the faulty use of terminology, but one of your criticisms might lead to more widespread use of a less logical alternative. The term 'post-tensioned concrete' is a contraction for concrete stressed by tensioning the wires (or bars) after casting the concrete, and correspondingly for pre-tensioned concrete. This contraction is consistent with the basic principle of prestressed concrete in that stressing of the concrete occurs before load is carried by the member, while tensioning (of the steel) is related to the time of casting. The term post-stressed concrete, which is often heard and may be encouraged by criticism of the alternatives mentioned, is purely an example of muddled thinking."

This letter would not have been written if the writer had used the word compressed instead of stressed. If we may attempt to follow the method of Socrates in seeking the truth, we may rewrite our correspondent's letter, in fewer words, as follows: "It is consistent with the principle of prestressed concrete to describe precompressed concrete as pre-tensioned (or post-tensioned) concrete. Any other term would be less logical." We then see that nothing could be more ridiculous than the use of the word tensioned to describe concrete that is compressed, whether pre or post the time when it has hardened. Anything less logical it is impossible to conceive. Contractions are useful devices, but if the only contraction that can be found conveys an entirely wrong meaning it is necessary to do without it if we wish to talk and write sensibly. No wonder we are continually hearing complaints that scientists and technologists do not express themselves in a way that can be understood by others.

In view of the issue in the near future of codes of practice for prestressed concrete in Great Britain and the U.S.A., some comments may be made on the terms at present in use. It may be thought to be hopeless at this stage to change the word prestressed to precompressed, but it is worth the effort. So it is suggested that, in the interests of true definition, the change be made in the codes; writers and designers will soon follow suit. It is true that the compression of the concrete may result in some tensile stress, but the fundamental idea is that the compression increases its resistance to tensile and shearing forces.

The term " at the time of transfer " or " at transfer " is used to denote the instant that the tensioning force is removed from the steel and the concrete is consequently put into a state of compression. This is unsatisfactory because it suggests that the tension in the wires is transferred to the concrete and that the concrete is thereby put into a state of tension. Instead of the contraction at transfer we should say " when the concrete is compressed " ; we use three more words, but remove all ambiguity. When we have saved three words a hundred times we have saved less than two minutes' talking time, or one minute's reading time. Ingenuity can often be misplaced.

The word stress should never be used unless the kind of stress is defined. Instead of the stress in the concrete we should say the compression, the tension, or the shearing in the concrete as the case may be. Instead of the stress in the steel we should say the tension in the steel. Instead of the top fibre and the bottom fibre of a member we should write the top and the bottom of a member—why the entirely unnecessary and misleading word fibre ?

The terms pre-tensioned steel and post-tensioned steel are both inaccurate. The affixes relate to the state of the concrete, not of the steel, when the steel is tensioned. In the interests of clarity of thought and at the expense of a few more words, we should say steel tensioned against hardened concrete, or steel tensioned in unset concrete ; again the contractions in common use are misleading, and a misleading contraction can have no place in clear thinking. To avoid confusion with ordinary reinforced concrete, tensioned steel should never be described as reinforcement. It should be called tensioned steel, and if any untensioned steel is used in precompressed concrete this should be called untensioned steel, leaving the word reinforcement to define only the steel in normal reinforced concrete.

It is also desirable to differentiate between (a) the abutments or mould-ends that resist the force of the jacks or other tensioning devices when the steel is stretched before the concrete has hardened, and (b) the wedges or other devices that secure the ends of the wires or bars, and so maintain the tension, when the stretching force is removed from concrete that is compressed after it has hardened. These very different things are both commonly described as anchorages. If abutments and fixings are not thought to be good enough, and a watery analogy is to be maintained, it would be better to describe as anchorages the means used for resisting the force of the stretching devices, and anchors to describe the wedges, cones, or other means of fixing the ends of the wires or bars.

As Chaucer wrote six hundred years ago, the " forme of speche is chaunge within a thousand year ", and new terms must be invented as science progresses, but it is doubtful if any previous generation has taken a word in common use, given it a directly opposite meaning, and currently used the same word to describe opposites, as has been done in the terminology of " prestressed concrete ".

Design of Statically-Indeterminate Structures by the "Plastic-Hinge" Method.

By R. GARTNER, D.Sc.(Eng.), M.I.Struct.E.

In the usual method of designing statically-indeterminate structures it is assumed that reinforced concrete is elastic and that the factor of safety is based on the yield-point of the steel and the plastic yield of the concrete. However, tests have shown that the effective factor of safety is higher if it is based upon the breaking strength of the member. In the plastic-hinge theory use is made of the breaking strength to reduce the higher bending moments at the supports or other places where high bending moments occur, thus enabling the reinforcement to be more easily arranged. Another advantage is that the solution of the simultaneous equations is simplified. For these reasons the method may well become increasingly important.

If a structure is loaded beyond its elastic limit it will first become plastic at points of maximum bending moment, which will eventually behave as hinges

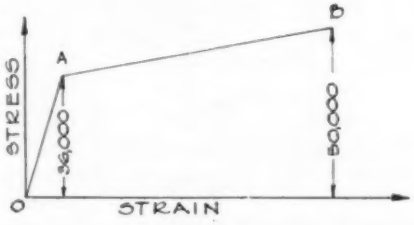

Fig. 1.

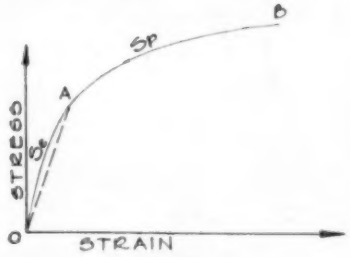

Fig. 2.

and are therefore called plastic hinges. The structure will collapse when so many plastic hinges develop that it becomes a mechanism. This may occur over the whole or part of the structure.

The "elastic" method of design is based on Hooke's law, which states that stress is proportional to strain. In the idealised stress-strain curve of mild steel (*Fig.* 1) the elastic part is OA. Point A, the limit of proportionality or yield-point, occurs at a stress of about 36,000 lb. per square inch. Beyond this the curve, corresponding to a state of plasticity, is very flat and the ultimate stress is about 50,000 lb. per square inch; in other words the steel is elongated considerably with little increase of load. The stress-strain curve of ordinary concrete is shown in *Fig.* 2. Concrete may be assumed to behave elastically as represented by the first part of the curve (s_c), being almost straight. Beyond this it is more curved, and some additional load can still be resisted before the concrete is crushed. The rotation a plastic hinge can withstand is determined by the "ultimate load" method, based on the distribution of strain and stress in the vertical section.

(1) Distribution of Strain.—Assuming that plane sections remain plane after bending (*Fig.* 3), $\frac{nd}{d-nd} = \frac{s_c}{s_s}$ and $n = \frac{s_c}{s_c + s_s}$ . . . . . (1)

in which s_c is the compressive strain in the concrete, s_s the tensile strain in the steel, and nd the depth of the neutral axis.

[*Note.*—The aim of this article is to derive safe formulæ for the rotation of hinges for statically-indeterminate structures and not to examine test results. Therefore idealised curves and critical rather than ultimate loads and deformations are used.]

In choosing the critical strain it should be borne in mind that cracks in the concrete under the critical load must not be too wide, and the concrete in compression should not collapse suddenly but visible cracks should appear when the critical load is reached. Further, the factor of safety should be such that under working load only hair-cracks develop. It is suggested that $s_{cp\ max.}$ = maximum strain of the plastic concrete = 0·2 per cent., the elastic strain s_{cE} (stress divided by E = 2,000,000 in.-lb.), the maximum strain of steel $s_{sp\ max.}$ = 1 per cent., and the elastic strain of steel s_{sE} (stress divided by E = 30,000,000 in.-lb.).

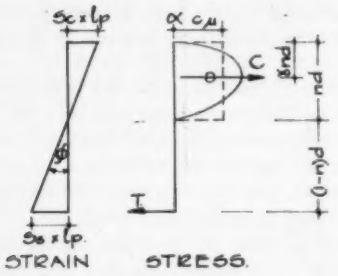

Fig. 3.

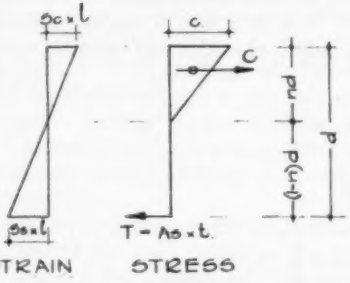

Fig. 3(a).

(2) Distribution of Stress.—The final distribution of stress in reinforced concrete members subjected to bending is similar at the critical load to that shown in *Fig.* 3. Two figures associated with the compressive part of this curve are important, namely (*a*) The total amount of the compressive force C. This can be expressed as $C = \alpha c_u bnd$ by replacing the curve by a rectangle (*Fig.* 3) and making $\alpha c_u = 0{\cdot}45 - 0{\cdot}6C_u$. ($C_u$ is the cube strength and b the breadth of the beam.) (*b*) The depth of the centre of gravity γnd, where γ is about 0·4. These are average values and must be adapted to special cases.

The Rotation of a Hinge.

To be able to calculate its rotation, the type of plastic hinge must be known. This depends on the amount of steel. With a small percentage (under-reinforced) the steel will become fully plastic before the concrete (a steel hinge); with a large amount of steel the concrete will first be fully plastic (concrete hinge). A balanced design is one in which both materials become plastic at the same time. At the critical load (*Fig.* 3) the total compression C is $\alpha c_u bnd$ and the total tension T is $\frac{p_s}{100}.t_u bd$, in which t_u is the ultimate stress in the steel, and p_s is the percentage of steel. $C = T$, and

$$n = \frac{p_s}{100}.\frac{t_u}{\alpha c_u} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

From equations (1) and (2),

$$p_s = \frac{s_c}{s_c + s_s} \cdot \frac{100\alpha c_u}{t_u} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

This is the limiting value, if ultimate stresses are used.

Steel Hinges (Bending Only).

The development of a plastic hinge in the steel in concrete reinforced with round mild steel bars can be described as follows. The maximum elastic moment M_E (*Fig.* 4) occurs at the yield point. The stress is still proportional to the strain. If the load is increased the steel becomes plastic at this point and the plasticity will extend to the neighbouring sections. The stress in the steel in the plastic part increases gradually; therefore, as the moment increases

Fig. 4.

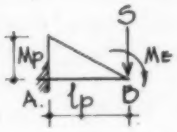

Fig. 5.

with further loading, the neutral axis rises and the stress in the concrete increases. Eventually the concrete also becomes plastic and will be crushed due to a moment M_p.

For a moment M_E, when yielding begins (*Fig.* 3a),

$C = \frac{1}{2}bdnc$ (c_c is the stress in the concrete).

$T = \frac{p_s}{100} t_y . bd$ (t_y is the tension in the steel at the yield point).

$$C = T; \text{ therefore } n = \frac{p_s t_y}{50 c_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

EXAMPLE 1.—The following values are assumed. Critical stress = 0·95 per cent. of the actual ultimate stress. $t_y = 36{,}000$ lb. per square inch. $0{\cdot}95 t_u = 0{\cdot}95 \times 50{,}000 = 47{,}500$ lb. per square inch; $E_s = 30{,}000{,}000$ lb. per square inch; αc_u, $0{\cdot}5 \times 0{\cdot}95 \times 5250 = 2500$ lb. per square inch; $E_c = 2{,}000{,}000$ lb. per square inch; $p_s = 0{\cdot}8$ per cent.; $s_{cp} = 0{\cdot}2$ per cent.; $s_{sp} = 1$ per cent.

For a balanced design, from (3),

$$p_s = \frac{0{\cdot}2}{0{\cdot}2 + 1} \times \frac{100 \times 0{\cdot}5 \times 5250}{50{,}000} = 0{\cdot}88 \text{ per cent.}$$

Therefore, with $p_s = 0{\cdot}8$ per cent., a steel hinge will develop. From (4)

$$n = \frac{0{\cdot}8 \times 36{,}000}{50 c_c} = \frac{576}{c_c} \quad . \quad . \quad . \quad (4a)$$

(*a*) For the maximum elastic moment M_E (*Fig.* 4):

From (1), $$n = \frac{\frac{c}{E_c}}{\frac{c}{E_c} + \frac{t_y}{E_s}} = \frac{c}{c + \frac{E_c}{E_s}.t_y} = \frac{c}{c + \frac{36{,}000}{15}} = \frac{c}{c + 2400}.$$

From (4a), $$\frac{576}{c} = \frac{c}{c + 2400}.$$

Therefore $c^2 = 576c + 1{,}380{,}000$, from which $c = 1497$ and

$$n = \frac{576}{1497} = 0{\cdot}384.$$

$$M_E = T\left(1 - \frac{n}{3}\right)d = \frac{0{\cdot}8}{100} \times 36{,}000\left(1 - \frac{0{\cdot}384}{3}\right)bd^2 = 251bd^2.$$

(*b*) For a maximum plastic moment M_p (*Fig.* 4), from (2),

$$n = \frac{0{\cdot}8}{100}.\frac{47{,}500}{0{\cdot}5 \times 5000} = 0{\cdot}151, \text{ and with } \gamma = 0{\cdot}4 \text{ (Fig. 3)},$$

$$M_p = Td(1 - \gamma n) = \frac{bd^2}{100} \times 0{\cdot}8 \times 47{,}500(1 - 0{\cdot}4 \times 0{\cdot}151) = 355bd^2$$

$$\Delta M = M_p - M_E = bd^2(355 - 251) = 104bd^2.$$

Let A be the peak of the bending moment curve, AB $= l_p$ (the length of the plastic part), and S the shearing force at point B (*Fig.* 5). Then

$$M_E + Sl_p = M_p \text{ or } l_p = \frac{M_p - M_E}{S} = \frac{\Delta M}{S}.$$

As l_p is small the shearing force can, as a first approximation, be assumed to be equal to the shearing force at A and the load on l_p can be neglected.

The strain s_s may be assumed to be uniform over l_p as the stress is nearly uniform. Therefore the plastic strain is $s_s l_p$ and the angle of rotation of the hinge (*Fig.* 3) is

$$\phi = \frac{s_s l_p}{(1 - n)d} \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Concrete Hinges.

The development of a concrete hinge can be visualised as follows. When the limit of elasticity is reached (part OA, *Fig.* 2), a moment M_E will develop at the point of maximum bending moment. If the load is increased the concrete will become plastic, that is the stresses across a vertical section will no longer vary linearly but the stress line will become a curve (*Fig.* 3), and as the load increases the plasticity will extend to the neighbouring parts (*Fig.* 4). The difference between the plastic and elastic strain in concrete is much smaller than that of steel. On the other hand, the stress-strain curve of concrete is still rising in part s_p (*Fig.* 2). Therefore the bending moment will increase over a greater distance l_p (*Fig.* 4) than in the case of a steel hinge. Hence the term plastic hinge applies only loosely and could perhaps better be called a plastic zone.

A general formula for n is derived as follows. c_E is the yield stress of concrete; t is the stress in the steel (*Fig.* 3a), $C = c_E bd\frac{n}{2}$, $T = t\frac{p_s}{100}.bd$, and $C = T$.

Therefore $t = 50c_E\frac{n}{p_s}$ Also $n : (1 - n) = \frac{c_E}{E_c} : \frac{t}{E_s}$ or $\frac{n}{(1-n)} = \frac{p_s}{50n} \times m$,

$$\left(m = \frac{E_s}{E_c}\right) \quad \text{and} \quad 50n^2 = p_s m - p_s m.n.$$

Therefore $n = \sqrt{\frac{p_s m}{50} + \left(\frac{p_s m}{100}\right)^2} - \frac{p_s m}{100}$.

Assuming that $p_s = 1{\cdot}5$ per cent. and $m = \frac{E_s}{E_c} = 15$,

$$n = \sqrt{0{\cdot}45 + 0{\cdot}0506} - 0{\cdot}225 = 0{\cdot}4824, \text{ and } t = 50\frac{0{\cdot}4824}{1{\cdot}5}c_E = 16{\cdot}08c_E.$$

EXAMPLE II.—$c_E = 1400$ lb. per square inch; $c_u = 3000$ lb. per square inch; $p_s = 1{\cdot}5$; $m = 15$; $\alpha = 0{\cdot}6$; $\gamma = 0{\cdot}4$; $t_u = 50{,}000$ lb. per square inch.

(a) For the maximum elastic moment M_E (*Fig.* 3a),

$$n = 0{\cdot}4824, \; t = 16{\cdot}08 \times 1400 = 22{,}500.$$

$$C = 1400\frac{0{\cdot}4824}{2}bd = 337{\cdot}4bd. \quad M_E = 337{\cdot}4\left(1 - \frac{0{\cdot}4824}{3}\right)bd^2 = 283bd^2.$$

(b) For the maximum plastic moment M_p (*Fig.* 3), assuming critical stresses of $0{\cdot}95c_u$ and $0{\cdot}95t_u$:

$$C = \alpha 0{\cdot}95c_u \times bdn = 0{\cdot}6 \times 0{\cdot}95 \times 3000bdn = 1710bdn,$$

$$T = \frac{p_s}{100}bd \; 0{\cdot}95t_u = \frac{1{\cdot}5}{100}bd \; 0{\cdot}95 \times 50{,}000 = 712bd.$$

$$C = T \text{ and } n = \frac{712}{1710} = 0{\cdot}417. \quad M_p = 712bd^2(1 - 0{\cdot}4 \times 0{\cdot}417) = 593bd^2.$$

Therefore $\Delta M = M_p - M_E = (593 - 283)bd^2 = 310bd^2$, and $l_p = \frac{\Delta M}{S}$.

Assuming that the strain at the compressive edge s_c is uniform over the length l_p (*Fig.* 3),

$$\phi = \frac{s_c l_p}{nd} \quad . \quad . \quad . \quad . \quad . \quad . \quad (5a)$$

From (5) and (5a) it is possible to calculate the angle of rotation of a plastic hinge.

The question now arises whether the calculation of ϕ is necessary or, as in the case of structural steel, it can be ignored. In other words, do the plastic hinges, which develop at certain loads, work like real hinges? Is the plastic length l_p very small and the angle ϕ not restricted? These two assumptions are examined by means of an example, namely a fully-fixed beam with a uniformly-distributed load. For comparison, the same example will be examined for a beam of structural steel. For the sake of simplicity a rectangular section is chosen.

(1) FULLY-FIXED RECTANGULAR BEAM OF STRUCTURAL STEEL (*Fig.* 6).—

Such a beam will fail when the fixing and maximum moments in the span are sufficient to cause plasticity. If there is no restriction of the rotation of the hinge these three moments will be equal. Therefore

$$M_p = -M_s = M_m = \tfrac{1}{16}.wL^2 \quad . \quad . \quad . \quad . \quad (6)$$

The angle of rotation does not appear in this equation, but can be found as follows.

(a) At the limit of elasticity (*Fig.* 7),

$$M_E = f_y.\tfrac{1}{2}.\frac{d}{2}.b.\tfrac{2}{3}d = f_y.b.\frac{d^2}{6} \quad . \quad . \quad . \quad . \quad (7)$$

Also

$$M_E = \frac{I}{x}.wL^2 \quad . \quad . \quad . \quad . \quad . \quad (8)$$

(b) At the plastic stage.—The distribution of stress at the plastic stage is shown in a simplified form in *Fig.* 8.

$$M_p = \alpha f_u.\frac{d}{2}.b.\frac{d}{2} = \alpha f_u\frac{bd^2}{4} \quad . \quad . \quad . \quad . \quad (9)$$

For the sake of simplicity, assume that $\alpha f_u = f_y$; then

$$M_p = f_y\frac{bd^2}{4} \quad . \quad . \quad . \quad . \quad . \quad (9a)$$

From (6) and (9a), $wL^2 = 4f_ybd^2$, and, inserting this in (7) and (8),

$$\frac{I}{x}.4f_ybd^2 = f_y\frac{bd^2}{6}.$$

Therefore $x = 24$, that is $M_E = \tfrac{1}{24}wL^2$ . . . . (10)

$$\Delta M = M_p - M_E = (\tfrac{1}{16} - \tfrac{1}{24})wL^2 = \tfrac{1}{48}wL^2 \text{ and } S = \frac{wL}{2}.$$

Therefore

$$l_p = \frac{\Delta M}{S} = \frac{L}{24}.$$

This result shows that the plastic length cannot be regarded as very small. If the load that the beam can carry at the elastic stage is w_1 (fully-fixed beam), $\tfrac{1}{16}wL^2 = \tfrac{1}{12}w_1L^2$; therefore $w_1 = \tfrac{3}{4}w$.

The load that may be carried after the supporting moments have become plastic is $w_2 = \tfrac{1}{4}w$ (freely-supported beam). Therefore the moments within the span are:

Elastic stage, $M_1 = \tfrac{1}{24}.\tfrac{3}{4}wL^2 = \tfrac{1}{32}wL^2$.

Plastic stage, $M_2 = \tfrac{1}{8}.\tfrac{1}{4}wL^2 = \tfrac{1}{32}wL^2$. Together these are $\tfrac{1}{16}.w.L^2$, which agrees with the original values.

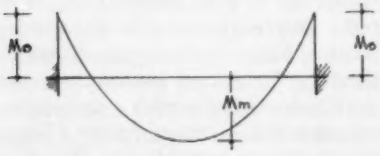

Fig. 6.

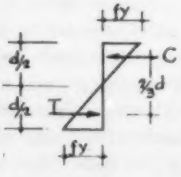

Fig. 7.

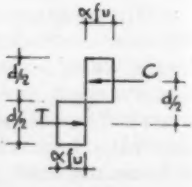

Fig. 8.

The angle of rotation at the supports due to the plasticity can now be calculated.

For a fully-fixed beam it is $\phi = 0$. For a freely-supported beam it is

$$\phi = \frac{1}{3}.\frac{1}{32}.wL^3.\frac{1}{EI} = \frac{1}{96}\frac{wL^3}{EI} \quad . \quad . \quad . \quad . \quad (11)$$

As (10) represents the maximum elastic moment at the yield point,

$$f_y = \frac{M_E d}{I.2} = \frac{1}{48}.\frac{wL^2 d}{I} \quad \text{or} \quad wL^2 = \frac{48 f_y I}{d}.$$

Inserting in (11),

$$\phi = \tfrac{1}{2}\frac{f_y L}{Ed} = \tfrac{1}{2}\frac{s_E}{d}L \quad . \quad . \quad . \quad . \quad (12)$$

According to (5a), however,

$$\phi = \frac{2 s_p l_p}{d}. \quad . \quad . \quad . \quad . \quad . \quad (13)$$

as $n = 0{\cdot}5$ for a rectangular section.

Equating (12) and (13), $\frac{1}{2}.\frac{s_E}{d}.L = \frac{2 s_p l_p}{d}$, so that

$$s_p = \frac{L}{4}.s_E\frac{1}{l_p} = \frac{L}{4}s_E.\frac{24}{L} = 6s_E = 6 \times \frac{36{,}000 \times 100}{30{,}000{,}000} = 0{\cdot}72 \text{ per cent.},$$

which is not too great for structural steel.

(2) Reinforced Concrete Beam with a Steel Hinge.—A similar beam of reinforced concrete such that a "steel hinge" develops. Again

$$M_p = -M_s = M_m = \tfrac{1}{16}wL^2 \quad . \quad . \quad . \quad . \quad (6)$$

Using the data of Example I, the maximum elastic moment $= 251bd^2 = \frac{1}{x}wL^2$.
The maximum plastic moment $= 355bd^2 = \frac{1}{16}wL^2$.

Therefore $x = 22{\cdot}6$, or $M_E = \frac{1}{22{\cdot}6}wL^2 \quad . \quad . \quad . \quad . \quad (14)$

$$\Delta M = \left(\frac{1}{16} - \frac{1}{22{\cdot}6}\right)wL^2 = \frac{1}{54{\cdot}8}wL^2, \text{ and } S = \frac{wL}{2}.$$

Therefore $l_p = \dfrac{\Delta M}{S} = \dfrac{L}{27{\cdot}4}$.

From (11), $\phi = \dfrac{1}{96}\dfrac{wL^3}{EI}$. . . . . (15)

From (14), $f_y = \dfrac{M_E}{I}(1 - n)d\dfrac{E_s}{E_c}$ where $(1 - n) = (1 - 0{\cdot}384) = 0{\cdot}616$.

$$f_y = \frac{wL^2}{22{\cdot}6} \times 0{\cdot}616\frac{E_s}{E_c}\frac{d}{I}.$$

Therefore $L^2 = \frac{36{\cdot}7 f_y E_c I}{E_s d w} = 36{\cdot}7 s_{ES} E_c \frac{I}{dw}$, and substituting in (15),

$$\phi = 0{\cdot}382 . s_{ES} . \frac{L}{d} \quad . \quad . \quad . \quad . \quad . \quad (16)$$

From (5), $\phi = \frac{s_{ps} l_p}{(1 - n)d}$, where $n = \frac{0{\cdot}384 + 0{\cdot}151}{2} = 0{\cdot}268$ (the average value), so that

$$\phi = \frac{s_{ps}}{0{\cdot}732 d} . \frac{L}{27{\cdot}4} = \frac{s_{ps}}{20} . \frac{L}{d}.$$

Equating this to (16), $0{\cdot}382 s_{ES} \frac{L}{d} = \frac{s_{ps} L}{20d}$; therefore $s_{ps} = 7{\cdot}64 s_{ES}$.

$$s_{ps} = 7{\cdot}64 \times \frac{36{,}000 \times 100}{30{,}000{,}000} = 0{\cdot}916 \text{ per cent.}$$

and $$s_{pc} = 0{\cdot}916 \frac{0{\cdot}151}{0{\cdot}849} = 0{\cdot}163 \text{ per cent.}$$ s_{ps} is rather high.

(3) CONCRETE BEAM WITH A CONCRETE HINGE.—A beam as in (2) but with a concrete hinge. Again

$$M_p = -M_s = M_m = \tfrac{1}{16} w L^2 \quad . \quad . \quad . \quad . \quad (6)$$

Using the data of Example II,

(a) the maximum elastic moment $M_E = 283 b d^2 = \frac{1}{x} w L^2$.

(b) the maximum plastic moment $M_p = 593 b d^2 = \frac{1}{16} w L^2$.

From which $$x = 34, \; M_E = \tfrac{1}{34} w L^2 \quad . \quad . \quad . \quad . \quad (17)$$

$\Delta M = \left(\frac{1}{16} - \frac{1}{34}\right) w L^2 = \frac{1}{30{\cdot}2} w L^2$ and $S = \frac{wL}{2}$. Therefore $l_p = \frac{\Delta M}{S} . \frac{L}{15{\cdot}1}$.

As before, $$\phi = \frac{1}{96} . \frac{wL^3}{EI} \quad . \quad . \quad . \quad . \quad (15)$$

From (17), $f_{Ec} = \frac{wL^2}{34I} \times 0{\cdot}482 d$, and $wL^2 = 70{\cdot}5 f_{EC} \frac{I}{d}$. Substituting in (15),

$$\phi = 0{\cdot}735 \frac{f_{EC} I L}{EId} = 0{\cdot}735 s_{EC} \frac{L}{d} \quad . \quad . \quad . \quad . \quad (18)$$

From (5a), assuming an average value of $n = 0{\cdot}45$,

$$\phi = \frac{s_{pc}}{0{\cdot}45 d} . \frac{L}{15{\cdot}1} = \frac{s_{pc}}{6{\cdot}8} \frac{L}{d}.$$

Equating this to (18), $0{\cdot}735 s_{EC} \frac{L}{d} = \frac{s_{pc}}{6{\cdot}8} . \frac{L}{d}$. Hence $s_{pc} = 5 . s_{EC}$

or $$s_{pc} = 5 \times \frac{1400 \times 100}{2{,}000{,}000} = 0{\cdot}35 \text{ per cent.}$$

This is too high, and the plastic moment should be increased so that w_1 becomes greater and w_2 smaller.

Summary.

These three examples show that: (1) The length of the plastic hinge is not very small, which possibly will not matter very much. (2) The strain in the structural steel example is not too high, so that the rotation need not be taken into account, but it is advisable to check this angle in extreme cases. (3) The strain in a steel hinge is rather large and it is advisable to take account of the angle of rotation. (4) The strain in a concrete hinge is large and the angle of rotation cannot be ignored.

Calculation of Unknown Quantities.

It is best to choose the maximum moments as unknowns X_1 to X_n. Their calculation is similar to that for elastic conditions. There are as many equations as unknowns. Each equation in the elastic stage expresses the fact that the angle of rotation of points 1 to n must remain zero, while in the plastic stage these angles equal the rotation of the plastic hinges 1 to n, that is δ_1 to δ_n.

The equations for the plastic state are easier to solve than those for the elastic state, as probable values of the unknowns X_p can be chosen and inserted into the equations. So long as δ_1 to δ_n on the right-hand side have the same sign as δ_{01} to δ_{0n} and are between zero and the maximum allowable rotation ϕ, their values are permissible.

Approximate Calculation of Unknown Quantities.

The equation for the unknown, X_p, in the elastic case with rigid supports is

$$\delta_{0r} + X_1\delta_{1r} + \ldots X_{r-1}.\delta_{r-1r} + X_r.\delta_{rr} + X_{r+1}.\delta_{r+1,r} + \ldots X_n\delta_{nr} = 0 \quad . \quad . \quad (19)$$

If the structure is loaded above the elastic limit plasticity will develop, and, if the structure is designed in such a way that all plastic hinges develop simultaneously, (19) becomes

$$\delta_{0r}' + X_1\delta_{1r} + \ldots X_{r-1}.\delta_{r-1,r} + X_r\delta_{rr} + X_{r+1}\delta_{r+1,r} + \ldots X_n\delta_{nr} = \delta_r \quad . \quad . \quad (20)$$

X_1 to X_n remain constant at the ideal plastic stage, δ_{0r} becomes δ_{0r}', and the right-hand side changes from zero to δ_r.

Subtracting (20) from (19) $\delta_{0r}' - \delta_{0r} = \delta_r$, and writing for $\delta'_{0r} = r\delta_{0r}$, $\delta_{0r}(r-1) = \delta_r$. Writing $(r-1) = V$, then $V.\delta_{0r} = \delta_r$. This means that the inelastic rotation δ_r can be expressed as a percentage of the elastic rotation δ_{0r}.

The assumption that all the points 1 to n will become plastic at the same time is not always correct. However, only the unknowns near point r have an influence on X_r. The writer recommended a value of 0·3.*

Examples.

Bending moments when plasticity occurs should be checked for rotation of the plastic hinges and adjusted if necessary as shown in the following.

* See also "Design of Indeterminate Structures by the Plastic Method", "Concrete and Constructional Engineering" January and February 1953.

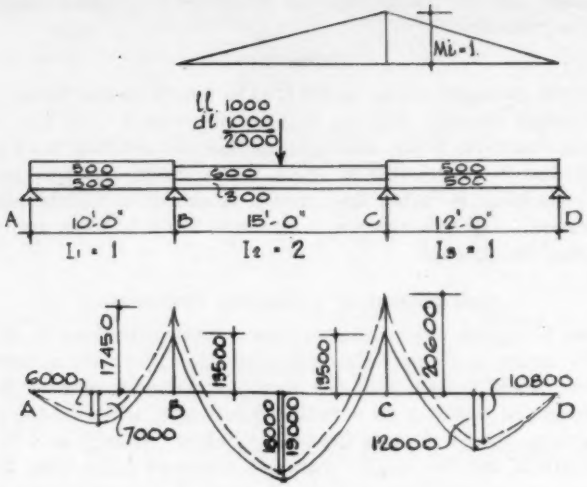

Fig. 9.

EXAMPLE III. CONTINUOUS BEAM.—The loads and the bending moments of a continuous beam of three spans are shown in *Fig.* 9. The full lines represent the moments at the plastic stage and the dotted lines the moments at the elastic stage.

(a) For a steel hinge at C, $M_p = -13{,}500$ ft.-lb. The angle to the left of C, when BC is regarded as a fully-loaded freely-supported beam, is, using the formula $EI\alpha = \int M_i M_{i2} ds$:

Case $M_i = 1$ is shown in *Fig.* 9.

$$\text{For the dead load, } EI_c\alpha_{c0}' = -\left[\frac{15^3}{24} \times \frac{600}{2} + \frac{15^2}{16} \times \frac{1000}{2}\right]$$

$$= -49{,}200$$

$$\text{For the live load, } EI_c\alpha_{c0}' = -\left[\frac{15}{24} \times \frac{300}{2} + \frac{15^2}{16} \times \frac{1000}{2}\right]$$

$$= -28{,}175$$

$$\text{Sum} = -77{,}375$$

The angle produced by M_p to the left of C is (from $M_B = M_C = -13{,}500$)

$$EI_c\alpha_{c2}' = \int M_i M_p ds = \left(\frac{15}{6 \times 2} + \frac{15}{3 \times 2}\right) . 13{,}500 = +50{,}600;$$

sum $-77{,}375 + 50{,}600 = -26{,}775$.

The angle to the right of C, when CD is regarded as a fully-loaded free beam, is

$$\text{Due to dead load, } EI_{cac0}'' = -\left(\frac{12^3}{24} \times 500\right) = -36{,}000$$

$$\text{Due to live load, } EI_{cac0}'' = -\left(\frac{12^3}{24} \times 500\right) = -36{,}000$$

$$-72{,}000$$

The angle produced by M_p is

$$EI_{cac}'' = \frac{12}{3} \times 13{,}500 = +54{,}000\text{; sum } -72{,}000 + 54{,}000 = -18{,}000.$$

The total angle is

$$EI_c\phi = -26{,}775 - 18{,}000 = -44{,}775 \quad . \quad . \quad . \quad (21)$$

For BC, $I = 2$; for CD, $I = 1$. Therefore no plasticity will be caused in BC and the whole angle ϕ will derive from the plasticity in CD. The shearing force at the end of the plastic hinge in C for beam CD is approximately equal to the shearing force at C. This, for the critical load with a factor of safety of 1·75, is

$$S = R_c = 1{\cdot}75\left(\frac{12}{2} \times 1000 + \frac{13{,}500}{12}\right) = 12{,}500 \text{ lb.}$$

Using the constants and stresses in Example I, $\Delta M = 104bd^2$.

Therefore $l_p = \dfrac{104bd^2}{12{,}500} = 0{\cdot}0083bd^2$. According to equation (5),

$$\phi = \frac{s_s l_p}{(1-n)d}, \text{ and, if } n = 0{\cdot}268,\ \phi = \frac{s_s 0{\cdot}0083bd^2}{0{\cdot}732d} = s_s \times 0{\cdot}0113bd.$$

But from (19) ϕ is also

$$\frac{44{,}775 \times 144}{EI_c} 1{\cdot}75 = \frac{44{,}775 \times 144 \times 12 \times 1{\cdot}75}{2{,}000{,}000bd^3} = \frac{67{\cdot}5}{bd^3}.$$

Therefore $s_s \times 0{\cdot}0113bd = \dfrac{67{\cdot}5}{bd^3}$.

Assume that $b = 6$ in., $d = 12$ in., $s_s = 0{\cdot}008 = 0{\cdot}8$ per cent. (which is acceptable), and $s_c = \dfrac{0{\cdot}268}{0{\cdot}732} \times 0{\cdot}8 = 0{\cdot}293$ per cent. (which is high).

(b) For a concrete hinge.—Use the values and constants in Example II and the angle of rotation as in (a). $\Delta M = 310bd^2$. With a factor of safety of 2,

$$S = R_c = 2\left(\frac{12}{2} \times 1000 + \frac{13{,}500}{12}\right) = 14{,}300.$$

$$l_p = \frac{310bd^2}{1430} = 0{\cdot}0217bd^2.\quad \phi = \frac{s_c l_p}{nd}.$$

If $n = 0{\cdot}45$, $\phi = \dfrac{s_c \times 0{\cdot}0217bd^2}{0{\cdot}45d} = s_c \times 0{\cdot}0482bd.$

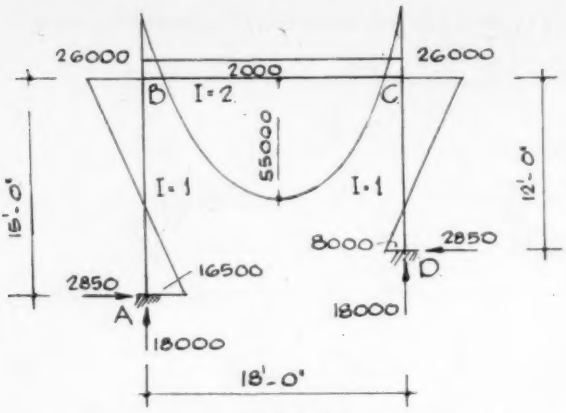

Fig. 10.

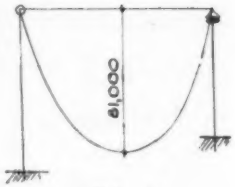

Fig. 11.

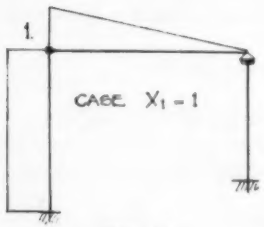

Fig. 12.

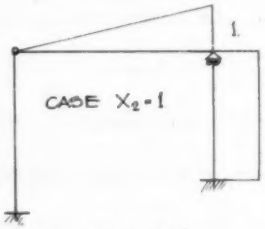

Fig. 13.

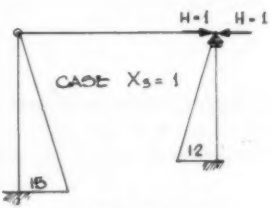

Fig. 14.

Also, $$\phi = \frac{44{,}775 \times 144 \times 2 \times 12}{2{,}000{,}000 \times bd^3} = \frac{77}{bd^3}.$$

Therefore, $s_c \times 0{\cdot}0482bd = \frac{77}{bd^3}$ and, assuming that $b = 6$ in. and $d = 12$ in., $s_c = 0{\cdot}0021 = 0{\cdot}21$ per cent., which is acceptable.

EXAMPLE IV.—A RIGID FRAME.—The bending moments at the plastic stage of a rigid frame supporting on the beam a uniform load of 2000 lb. per foot are shown in *Fig.* 10. Check the rotation at B ($M_p = 26{,}000$). The bending moments of the determinate case are shown in *Fig.* 11. The cases $X_1 = 1$, $X_2 = 1$, and $X_3 = 1$ are shown in *Figs.* 12, 13, and 14.

The rotation to the right of B in the determinate case is

$$\delta_{01}^{BC} = -\frac{18}{3 \times 2} \times \frac{18^2}{8} \times 2000 = -243{,}000.$$

The rotation produced by $M_p = 26{,}000$ at B and at C is

$$\delta_{11}^{BC} + \delta_{12}^{BC} = \left(\frac{18}{3 \times 2} + \frac{18}{6 \times 2}\right)26{,}000 = +117{,}000.$$

The net rotation is $-243{,}000 + 117{,}000 = -126{,}000$.

At B in column AB for the determinate case δ_{10} is zero.

For $X_1 = 1$, the rotation is $\delta_{11}^{AB} = 15 \times 26{,}000 = 390{,}000$

For $X_3 = 2850$, the rotation is $\delta_{13}^{AB} = -\frac{15^2}{2} \times 2850 = -318{,}000$

$+ 72{,}000$

Therefore the total angle at B is $EI_c\phi = -126{,}000 + 72{,}000 = -54{,}000$.

No plasticity will develop in beam BC as its moment of inertia is double that of AB. The whole rotation will derive from column AB, point B.

(a) Steel Hinge.—The shearing force for the critical load, with a factor of safety of 1·75, is $S = H = 1{\cdot}75 \times 2850 = 5000$. Using the constants and stresses in Example I,

$$\Delta M = 104bd^2. \quad l_p = \frac{104bd^2}{5000} = 0{\cdot}021bd^2.$$

From (5), $\phi = \frac{s_s l_p}{(1-n)d}$.

If $n = 0{\cdot}268$, $\phi = \frac{s_s \times 0{\cdot}021bd^2}{0{\cdot}732d} = s_s \times 0{\cdot}0287bd$.

Also $\phi = \frac{54{,}000 \times 144}{EI_c} \times 1{\cdot}75 = \frac{54{,}000 \times 144 \times 1{\cdot}75 \times 12}{2{,}000{,}000 \times bd^3} = \frac{81{\cdot}5}{bd^3}$.

Hence $s_s \times 0{\cdot}0287bd = \frac{81{\cdot}5}{bd^3}$.

Assuming that $b = 9$ in. and $d = 14$ in., $s_s = 0{\cdot}00906 = 0{\cdot}906$ per cent., which is acceptable. $s_c = 0{\cdot}906 \times \frac{0{\cdot}268}{0{\cdot}732} = 0{\cdot}332$, which is high. $l_p = 37$ in.

(b) Concrete Hinge.—Using the values and constants of Example II and the angle of rotation as in (a), $\Delta M = 310bd^2$. Assuming a factor of safety of 2,

$$S = H = 2 \times 2850 = 5700 \text{ and } l_p = \frac{310bd^2}{5700} = 0{\cdot}0544bd^2.$$

Assuming that $n = 0{\cdot}45$, $\phi = s_c \frac{0{\cdot}0544bd^2}{0{\cdot}45d} = s_c \times 0{\cdot}123bd$. Also,

$$\phi = \frac{54{,}000 \times 144 \times 2 \times 12}{2{,}000{,}000bd^3} = \frac{93}{bd^3}.$$

$s_c \times 0{\cdot}123bd = \frac{93}{bd^3}$ and, assuming that $b = 9$ in. and $d = 13$ in.,

$$s_c = 0{\cdot}00033 = 0{\cdot}033 \text{ per cent., which is small.}$$

$$s_s = 0{\cdot}033 . \frac{0{\cdot}55}{0{\cdot}45} = 0{\cdot}04. \quad l_p = 83 \text{ in.}$$

The direct forces are small and can be neglected. If the direct forces have to be taken into account, the calculation of the plastic rotation can be made similarly by using the appropriate strains.

The last example shows that the plastic length l_p is rather large and would

be increased further if the direct forces were taken into account. In other words the action is more that of a plastic zone than a hinge.

In the last example the length l_p depends on c_E. Writing $c_E = 0{\cdot}6c_u = 1800$ instead of 1400 (as αc_u in Example II),

$$M_E = \frac{1800}{1400} \times 283bd^2 = 363bd^2 \text{ and } \Delta M = (593 - 363)bd^2 = 230\ bd^2,$$

so that l_p is reduced to $l_p' = \frac{230}{310} l_p = 0{\cdot}74 l_p$ or, with the dimensions of the last example, $l_p' = 61{\cdot}5$ in., which is nearly five times the depth d.

If instead of the approximate calculation with $\delta_r = V.\delta_{0r}$ a more exact calculation is desired, the values of ϕ_r have to be calculated from the approximate bending-moment curve and inserted in place of δ_r. The equation can now be solved in the same way as for the approximate calculations; ϕ can vary from zero to ϕ_{max}.*

The foregoing is an attempt to present briefly the main considerations in the calculation of plastic hinges. It should be stressed that the constants and stresses used have not been obtained as the result of tests and are purely illustrative.

* A quick solution in tabular form for highly-indeterminate structures is given in "The Ultimate Load Theory Applied to the Design of Reinforced and Prestressed Concrete Frames", by Professor A. L. L. Baker. (Concrete Publications, Ltd. Price 18s.)

New Road Bridge at Chiswick

A TWO-LEVEL road junction is to be constructed at Chiswick, London, W., where the Cromwell Road extension joins the Great West Road. The work is expected to cost £820,000 and to be completed in about two and a half years. The work will include a "fly-over" bridge about half a mile long, a bridge with four spans each 125 ft. long and 59 ft. wide between parapets, and a bridge 124 ft. wide between parapets to carry eight lines of traffic.

The bridges are to be constructed with prestressed precast concrete beams and reinforced concrete decks. The rest of the "fly-over" will be on embankments, with reinforced concrete retaining walls with brick facing. The works have been designed for the Ministry of Transport and Civil Aviation. The consulting engineer is Mr. Harry Brompton, B.Sc. (Eng.), M.I.C.E., and the consulting architect is Mr. George Stewart. The contractors are the Alderton Construction Co., Ltd.

Lifting a Heavy Slab.

WHAT is thought to be the heaviest slab so far raised by the lift-slab method is the roof of a factory at Lyndhurst, New Jersey, U.S.A. The slab weighs 1466 tons and has an area of 32,600 sq. ft. It was cast on the ground and raised to a height of 26 ft. by jacks on top of 24 steel columns spaced 40 ft. apart. Each jack exerted a force of nearly 75 tons.

In order to keep the weight of the slab low it was made with lightweight concrete. Fibre-board boxes 2 ft. 10 in. square by 1 ft. 1 in. deep were placed on the casting floor to form a slab 3 in. thick over the boxes with ribs projecting 1 ft. 1 in. below the underside. The concrete was specified to have a compressive strength of 3000 lb. per square inch at 28 days. The slab is fixed by welding the projecting lifting collars to steel plates welded to the columns. The slab was raised by the New England Lift-Slab Corporation.

Office Building in London.

FLOORS WITHOUT PROJECTING BEAMS.

A NEW office building (*Fig.* 1) in London, known as Clements House, has a main frontage to Gresham Street and subsidiary frontages to Wood Street and Milk Street. The site is roughly rectangular and has a total area of 29,800 sq. ft., of which 17,500 sq. ft. is occupied by the building. The Gresham Street wing (about 200 ft. by 50 ft.) and the Wood Street wing (about 150 ft. by 50 ft.) form an L-shaped building (*Fig.* 2). At basement level there is a courtyard, reached by ramps, for use as a car park.

The Gresham Street wing was built first and completed to a point 90 ft. from the corner of Wood Street, using a tower crane so placed as to cover the whole of this part. When the main structure of this section was completed the crane was moved so that it would cover the whole of the Wood Street wing. The finishing of the first part was carried out while the structure of the second part was being built.

The foundations are on the sandy

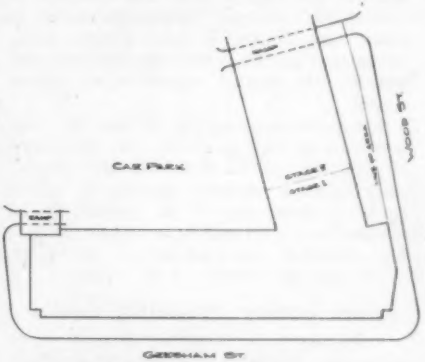

Fig. 2.—Plan of the Site.

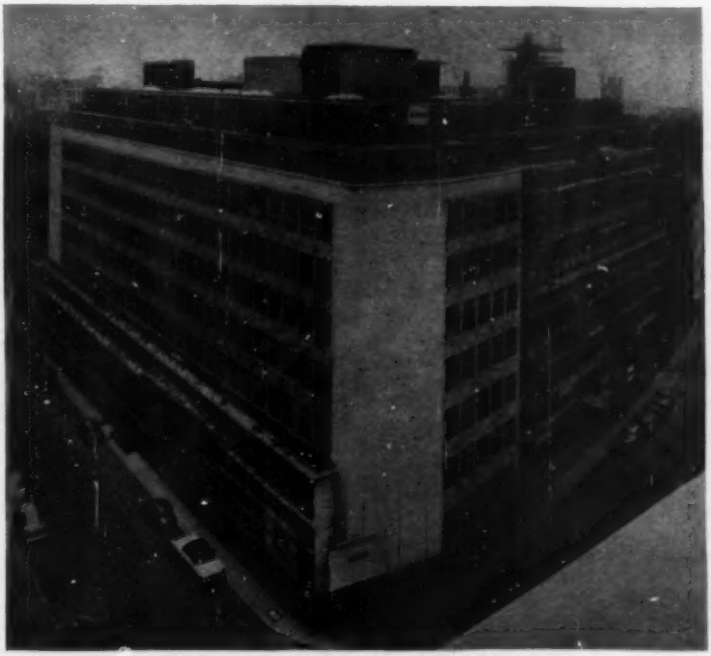

Fig. 1.—Clements House, Gresham Street.

ballast of the Reading Bed, and borings to a depth of 35 ft. showed that the depth of the ballast was sufficient for the load to be carried without the bearing exceeding 3 tons per square foot. The foundations are strips of concrete, about 15 ft. thick, which were more economical than isolated footings or shallower footings of reinforced concrete. Although seams of clay were found in the gravel beds, piling was not used because the load was widely distributed above the poorer strata.

A typical cross section of the building is shown in *Fig.* 3. There are two rows of columns, one on each side of a central corridor at a uniform spacing of 12 ft. 2½ in.; these are 18 in. square up to ground floor level and 14 in. square above. The external columns are 6 ft. 1¼ in. apart and are mostly 12 in. square.

Floors without Projecting Beams.

A feature of the building is the absence of beams projecting below the ceilings, thus providing freedom in dividing the floors. The floor slabs are of hollow-tile construction, spanning across the width, and are supported by reinforced concrete beam-strips within the depth of the slab. At the outside edges the beam-strips span between external columns, and along the corridors the beam-strips are supported on two parallel rows of columns as shown in *Figs.* 4 and 5. The slab is 9½ in. deep and is formed of 12-in. by 12-in. by 8-in. hollow clay tiles with 1½ in. topping of concrete. The ribs (*Fig.* 6) are 3¾ in. wide and reinforced with cold-worked twisted bars.

The loads for which the floor was designed are: Weight of slab, 68 lb. per square foot; live load, 50 lb. per square foot; partitions, 30 lb. per square foot; floor topping, 18 lb. per square foot; finishes, 9 lb. per square foot; total, 175 lb. per square foot. The arrangement of ribs for each bay was considered separately; slightly wider ribs were provided, 12 ft. 2½ in. apart, on the axis of the main columns.

Both top and bottom bars were cut and fixed in straight lengths; cranked bars were not used, thus saving labour. The bottom bars were wired to concrete spacers and the top bars were supported from the hollow tiles; both layers of reinforcement were wired to the reinforcement in the beam-strips.

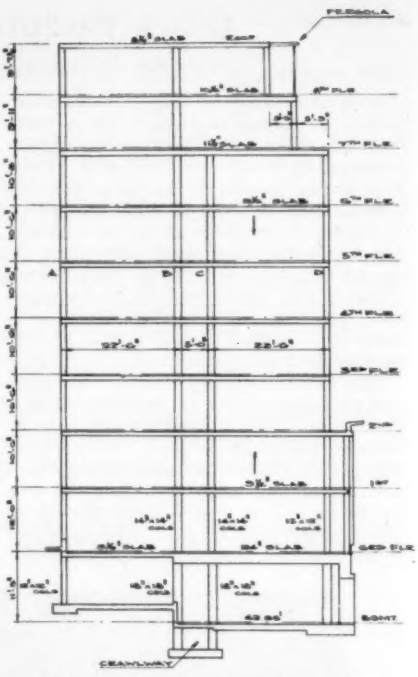

Fig. 3.—Section through Gresham Street Wing.

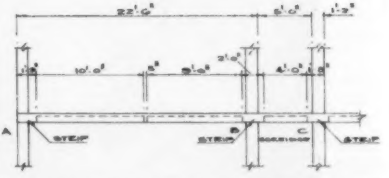

Fig. 4.—Typical Section through Floor.

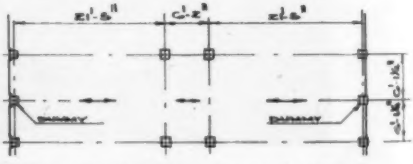

Fig. 5.—Typical Arrangement of Beams.

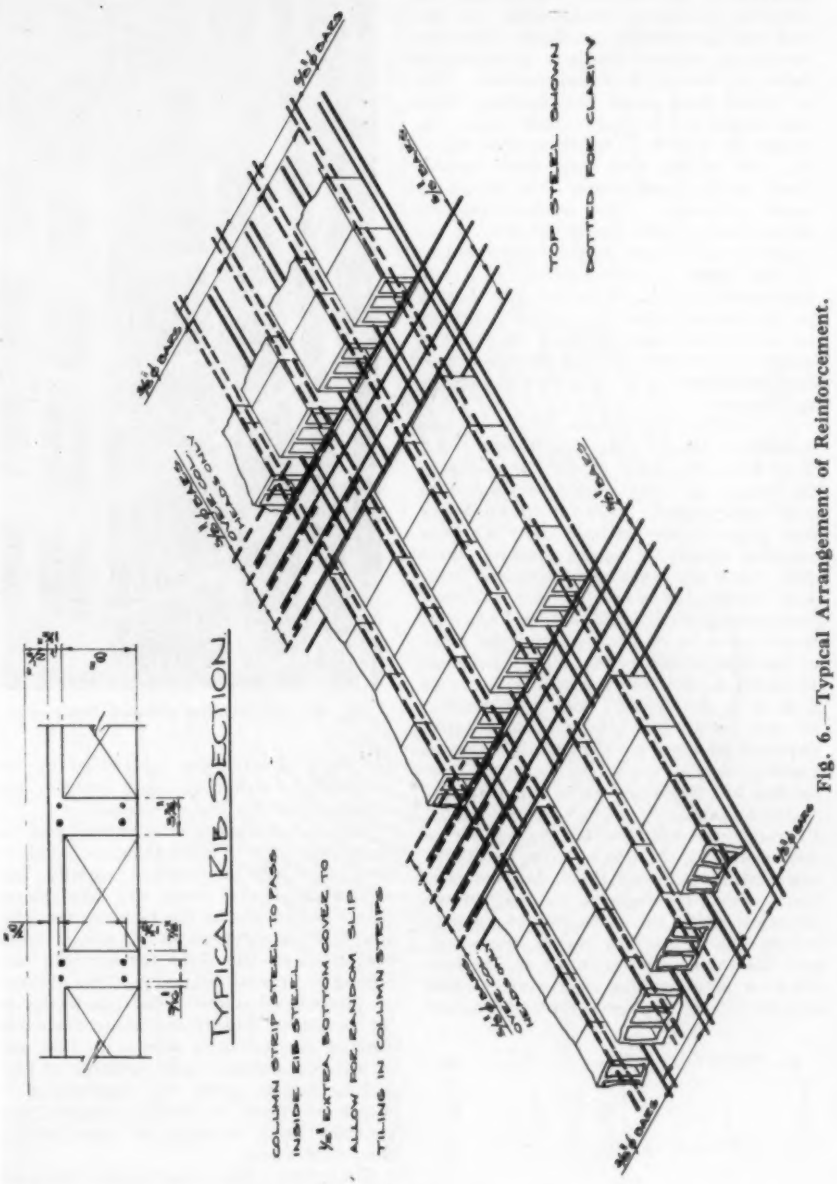

Fig. 6.—Typical Arrangement of Reinforcement.

BEAM-STRIPS.—These were designed as continuous rectangular beams and the bending moments calculated by the moment-distribution method. Shearing around the column heads is an important factor in this type of construction. For a typical floor panel the shearing stress was about 50 lb. per square inch. As shown in *Fig.* 6 no stirrups were used; the bars in the slab were fixed outside those in the beam-strips, thus providing some restraint. The beam-strips are nearly symmetrical about the head of the columns, and a part of the reinforcement in the beams passes through the reinforcement of the columns. The saving in stirrups resulted in a major economy in labour and material, and the relative weight of reinforcement in shallow beams compares favourably with that in projecting beams.

An alternative scheme (*Fig.* 7) was considered using projecting beams 18 in. deep by 9 in. wide across the width of the wings, and floor slabs 6 in. thick with 4-in. hollow tiles. Each rib would require one $\frac{3}{4}$-in. square twisted bar and the support beams A would contain three $\frac{7}{8}$-in. bars as main reinforcement with $\frac{1}{4}$-in. binders at 9-in. centres. To resist the shearing force at the supports the bars would have to be bent up and the pitch of the binders decreased. The tie-beams B would be nominally reinforced by two $\frac{1}{2}$-in. bars throughout, with $\frac{1}{4}$-in. binders at 9-in. centres. The weight of steel required for the flush hollow-tile floor was slightly less than 90 per cent. of that needed for this alternative scheme.

ADVANTAGES OF FLUSH HOLLOW-TILE FLOORS.—In addition to the saving of reinforcement, there is a saving in weight when compared with floors of solid concrete. Also, by spanning the floor in one direction only, the difficulty of keeping hollow tiles in position during concreting, and the increased weight of the hollow-tile floor with crossing ribs, were avoided and the labour of closing the ends of each

Fig. 7.—Alternative Arrangement of Beams

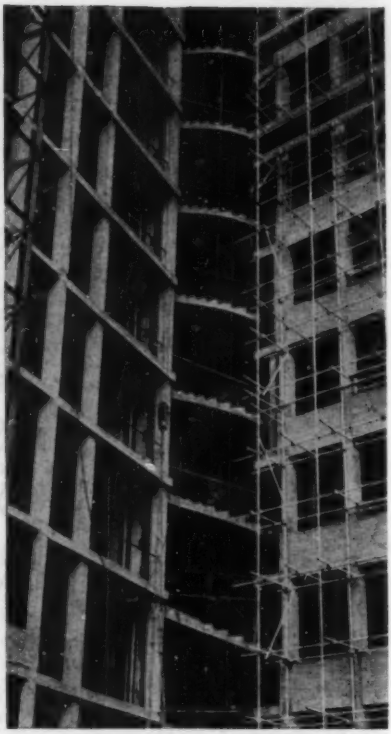

Fig. 8.—Horseshoe-shaped Staircase.

tile was saved. The simplicity of the shuttering resulted in rapid erection and economy in the use of timber.

The arrangement of reinforcement reduced the work of fabrication and enabled it to be put in position rapidly and accurately, since there are few places where congestion of the bars occurs; this simplified supervision at the site. Openings through the slabs for services presented no special difficulty. The absence of projections allowed the plastering to be completed rapidly and saved the extra cost of plastering in narrow widths and of forming arrises. The omission of projecting beams gives the impression of increased floor to ceiling height, and provides more freedom for sub-dividing the office areas.

COLUMNS.—The columns were designed to withstand the axial loads together with the additional bending stress due to the

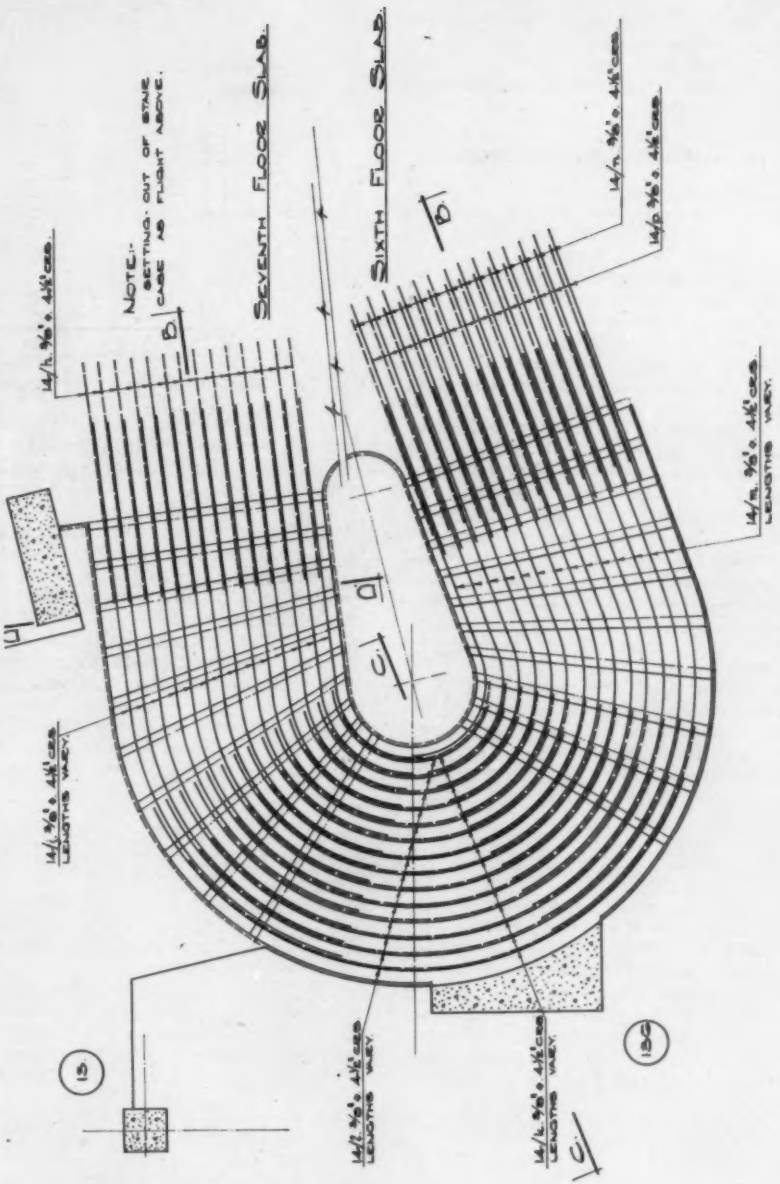

Fig. 9.—Plan of Staircase.

slabs. An aggregate with a maximum size of ¾ in. was used for the ribs and for the columns. The arrangement of bars at the connections between the columns and the floor slabs is straightforward and, in the absence of projecting beams, there is no congestion.

Horseshoe-shaped Staircase.

STAIRCASE.—The main staircase, at the rear of the entrance hall against the external wall at the junction of the two wings (*Fig.* 8), is horseshoe-shaped in plan. The half-landings are cantilevered from a single shaped column (*Fig.* 9), and the flights are supported at floor levels by wide shallow beams contained within the thickness of the slab. The thicknesses of the half-landings and the stair-waist are 6½ in. and 5 in. respectively.

The shuttering for the stair well comprised a timber drum, 18 ft. high, which was hoisted up the well as the work progressed. The drum was made in halves held 2 in. apart by wedges during concreting. The wedges were knocked out to release the shutters from the edges of the flights and landings, and the drum was then raised and wedged in its next position.

EXTERNAL PANEL WALLS.—Much of the external wall consists of precast concrete panels faced with quartzite slabs of random shape. As shown in *Fig.* 10, the panels rest on a supporting ledge and are held in position by cramps built into the 9-in. brickwork. Both the brickwork and the panels are supported on the edge of the structural slabs without projecting beams.

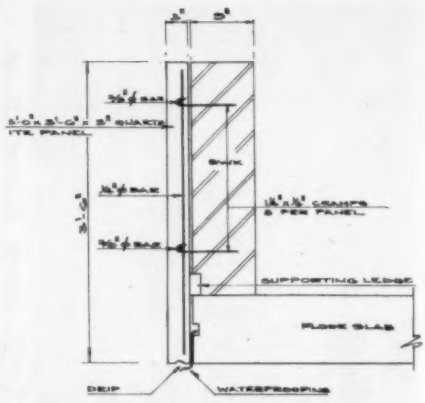

Fig. 10.—Method of Fixing Quartzite Panels.

Concrete was mixed in a 14/10 machine and distributed by the tower crane, using a ½-cu.-yd. skip, and placed at the rate of about 10 cu. ft. every three minutes. A cement silo of 20 tons capacity was installed on the site.

The building was completed about one year after the foundations were completed. The architects were Messrs. Trehearne & Norman Preston & Partners, the consulting engineers were Messrs. R. Travers Morgan & Partners, and the contractors were Messrs. Wates, Ltd.

Exhibition of Architecture in Berlin.

THE Joint Committee on Structural Concrete of the Cement and Concrete Association and the Royal Institute of British Architects are arranging two visits in connection with the architectural exhibition to be held in Berlin in September next. The first visit is from September 13 to 18 and the second from September 18 to 23. The programme includes visits to commercial, industrial, domestic, and other buildings under construction, the "Interbau" exhibition, the "City of To-morrow", and a building exhibition. Most of the buildings to be seen are of reinforced concrete, and precast concrete has been used extensively. Further information is available from the Association, 52 Grosvenor Gardens, S.W.1.

A Precast Cooling Tower.

At a gas works in Berlin-Mariendorf a hyperbolic cooling tower has been constructed of precast concrete units. The tower is about 100 ft. high, 80 ft. diameter at the bottom, and 47 ft. diameter at the top.

The structure (*Fig.* 2) consists of thirteen rings, with thirty-two horizontal members 10 in. wide and 3½ in. thick in each ring, supporting sixty-four diagonal members 12 in. wide by 3½ in. thick. Triangular precast slabs of reinforced concrete (*Fig.* 5), 1½ in. thick and of areas varying from 2 sq. yd. to 3¾ sq. yd., are supported on rebates formed in the ribs and horizontal members (*Fig.* 1). Each slab is secured by connecting bars at nine points, and the joints are filled with cement mortar. The cooling unit is supported by 132 columns, 8 in. square and 6 ft. 6 in. high.

The units were assembled from tubular-steel scaffolding and a tower crane inside the tower. The horizontal members for

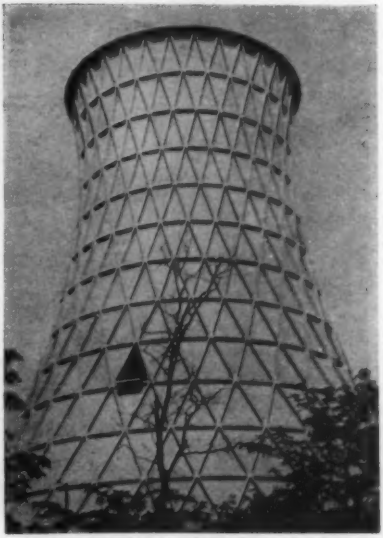

Fig. 2.—Cooling Tower Built of Precast Units.

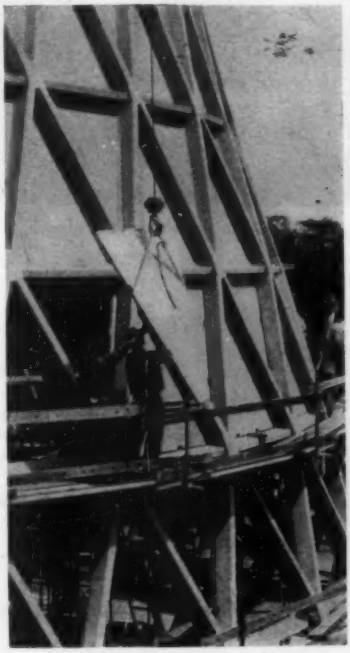

Fig. 1.—Erecting a Slab.

Fig. 3.—Detail of Joint shown in Fig. 4.

Fig. 4.—Erecting Diagonal Ribs.

Fig. 5.—A Triangular Slab.

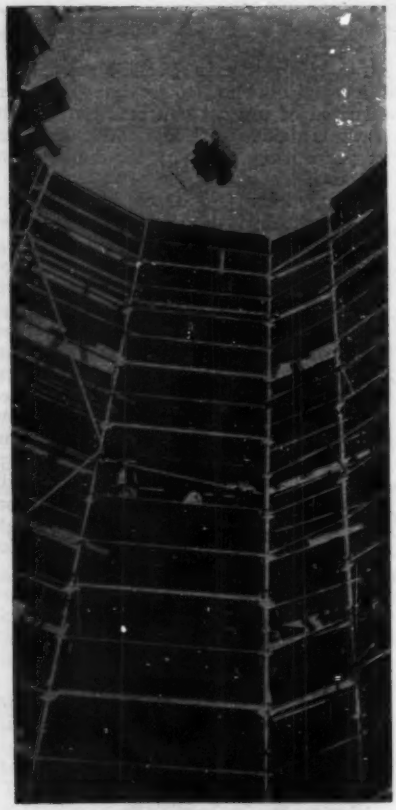

Fig. 6.—Interior Staging.

one ring were placed in position ; the ribs were then fixed together in pairs using steel cover-plates (*Figs.* 3 and 4), and were lifted into position. The cover plates also served as shuttering for the high-strength mortar used for the joints.

The erection of a complete ring required thirty-six hours. The crane was removed through gaps in the structure.

The tower was constructed in ten weeks, including the time required to make the precast units. The variation in the diameters of the horizontal rings was less than 0·1 per cent. (about $\frac{1}{2}$ in.) of the minimum diameter. The contractors were Beton- und Monierbau of Berlin.

Precast Shell Roofs.

By A. N. S. KULASINGHE, A.M.I.C.E.

An important factor in the cost of shell roofs is the shuttering, especially when it will be used only a few times. A method of reducing the cost of shuttering is to precast a roof in large sections (sometimes the whole shell) and erect them with vacuum pads so that allowable stresses are not exceeded during handling. The sections are joined with concrete cast in place, and the joints between the end frames and the edge-beams are made with concrete cast in place. This method has been used by Dr. Billner in the U.S.A. with apparent success.

The writer has developed a method of construction using small precast units assembled together by prestressing, and in which most of the precast units can be quickly made on a production line and handled by one man. The chief feature of the method is the assembly of the precast sections on the ground to form a series of tied arches which are later combined to form a monolithic shell which is jacked up to the required height, the columns being built up of precast units as the shell is raised.

This method was used for a roof of 50 ft. span and 20 ft. wide as a model (*Figs.* 1 and 2) for larger shells for quay-side warehouses. Details of the units are given in *Fig.* 3, and *Fig.* 4 shows the structure during erection.

The edge-beams, which are 3 ft. deep and 8 in. wide, were formed on the ground by assembling together precast sections each 3 ft. long. Four cables each of two 0·2-in. wires, at the corners of the cross section, were used to stress the sections together. Scaffold tubes of 2-in. diameter were then fixed between the edge-beams as ties to resist the horizontal thrust of the arches until the shell was made monolithic.

Fig. 1.

Two rails were then clamped on the ties and close to the edge-beams to carry a light travelling centering 2 ft. wide to support two arch-rings each 1 ft. wide; this centering could travel the full length of the shell. The unreinforced precast sections of the shell, 2½ in. thick, 12 in. wide, 3 ft. 3 in. long, were assembled on the centering and prestressed with 0·2 in. wires anchored in the edge-beams, producing a tied arch spanning between the edge-beams. The centering was then moved forward 2 ft. and the same procedure repeated until the full length of the shell was completed.

The precast end frames were then erected, and wires passed through holes formed in the segments to form continuous ducts between the end frames; these wires were tensioned and anchored to the end frames, starting from the crown and working towards the edge-beams on each side. This produced a tied arch of width equal to the span of the shell, and the tied arch was converted to a shell by tensioning the parabolic main cables (comprising twenty-four wires of 0·2 in. diameter) in the edge-beams while slackening the transverse ties between the edge-beams. This produced a self-supporting prestressed shell. The shell was jacked

Fig. 2.

to the height required, the columns being built as the shell was raised. The four walls were then built, the shell continuing to span between the columns.

This method also made it possible to arrange the orthogonal prestressing of the shell so as to prevent tensile stresses occurring due to dead and live loads. There was therefore no need to provide three layers of steel, which often make it difficult to ensure adequate cover and satisfactory concreting. The only mild steel reinforcement used was in the end frames (which were designed as Vierendeel trusses), in the edge-beams to resist stresses during lifting, and around openings in the shell. No special waterproofing treatment was used, and precasting enabled a good finish to be produced on the concrete. A coat of white cement was given to the outside to improve the reflection of heat. The roof

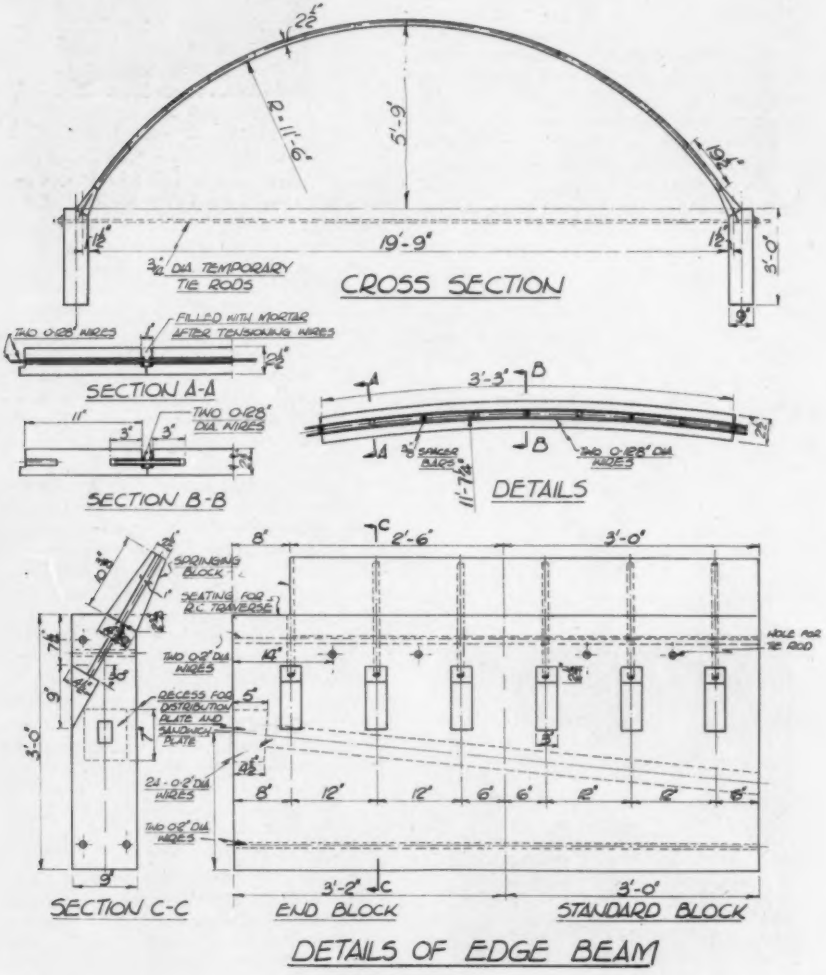

Fig. 3.

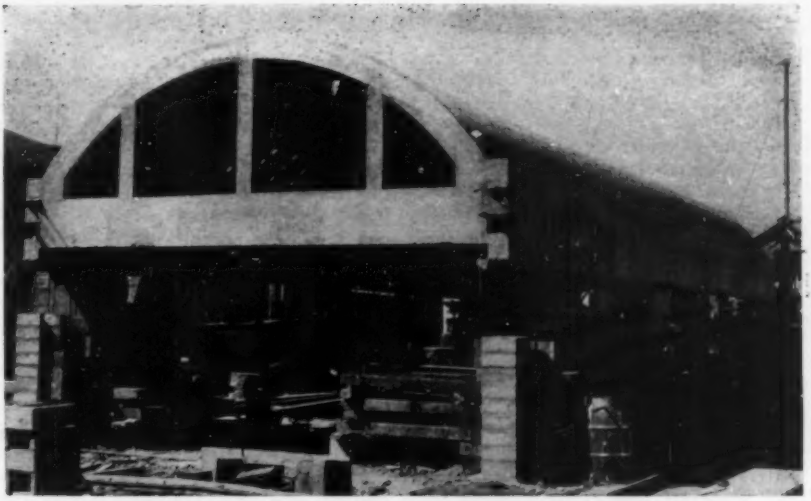

Fig. 4.—Precast Shell Roof during Construction.

has remained completely watertight in spite of the heavy rainfall in Colombo. There was also no need to provide thermal insulation for the temperatures prevalent in Colombo.

The advantages of this method are that (a) No shuttering or scaffolding is needed, (b) There is no difficulty in ensuring adequate cover to the steel, (c) There is no complicated arrangement of steel to be placed in several layers, (d) Factory production methods can be applied to the casting of the units, most of which can be handled by one man, (e) A high quality and finish can be obtained, (f) Work is done at ground level, and (g) There is a saving of steel.

The shell is designed by the usual methods, but is simplified because of the possibility of preventing cracking under working loads. The stresses resulting from the orthogonal prestressing during the assembly of the elements can be added to the stresses calculated for the shell. This method is applicable only to long shells due to the need for limiting the width to what is practicable during the "tied-arch" stage of the construction. Shells of 100 ft. span are easily built by this method; some problems relating to spans greater than 120 ft. are being investigated. A large warehouse has been planned using a large number of shells of 100 ft. span and about 30 ft. wide; these will be joined to form valleys between them. The precast units would be cast by the Colombo Port Commission.

The writer is Chief Assistant Harbour Engineer of the Colombo Port Commission, and is indebted to Mr. E. C. de Alwis, B.Sc.(Eng.), A.M.I.C.E., A.M.I.Mech.E., Harbour Engineer, for the facilities and help given to him, and to Mr. M. Chandrasoma, C.C.S., Port Commissioner, for permission to write this article.

Conference on Foundations in Frozen Ground.

A CONFERENCE on "Soil Engineering in Frost Areas" is to be held by the Massachusetts Institute of Technology from June 11 to June 21. Details of the arrangements and of the papers to be read may be had from Room 7-103, Massachusetts Institute of Technology, Cambridge 39, Mass., U.S.A.

Book Reviews.

"Applied Strength of Materials." By Alfred Jensen. (New York: McGraw-Hill Book Company, Inc. 1957. Price 43*s*.)

THIS book is clearly written, with only minor inexactitudes, and is suitable for students of Ordinary National Certificate standard. Its contents include the calculation of direct stresses and shearing stresses; the design of riveted and welded joints; torsion, shearing forces and bending moments on simply-supported beams, and the resulting stresses; the design of steel and timber beams; deflection of beams; continuous beams; stresses due to unsymmetrically applied loads; steel and timber columns, and notes on the properties of materials which are frequently used in engineering. Each chapter concludes with a summary which would be useful to students revising the work for examinations and a selection of illustrative problems and problems for solution. In some places the reasons for certain operations are not defined precisely, and in the chapter on materials the notes on cements and concrete could be improved. A feature of the book that may attract some students is that the use of the calculus has been avoided, although in some cases this has led to a statement of formulæ without giving derivations.—J. E. G.

"Grundbau-Taschenbuch." (Berlin: Wilhelm Ernst & Sohn. 1955. Vol. I, price 63 D.M. Vol. II, price 22 D.M.)

THE first part of Vol. I deals with soil mechanics, earth pressure, stability of foundations, distribution of pressure and settlement of elastic foundations, soil dynamics and its effect upon structures, and soil geology and biology. There are many tables for calculating earth pressure and 51 fully-worked examples. The second and third parts deal with materials, types of foundations, and methods of construction including sheet piles, cofferdams, direct bearing piles, caissons, retaining walls, and foundations subject to vibration and mining subsidence. This section gives methods of calculation for anchored sheet-pile walls and groups of piles. Vol. II gives the German regulations and codes of practice relevant to the subjects treated in the first volume. As each chapter has been contributed by a different author some overlapping has been unavoidable. Nevertheless, with its many tables and excellent illustrations, and mathematics well within reach of the average engineer, this is a useful book.

"Kempe's Engineers' Year Book for 1957." (London: Morgan Bros. (Publishers), Ltd. Two vols. Price £4 2*s*. 6*d*.)

THE sixty-second edition of this book, published in two volumes totalling nearly 3000 pages, contains substantial revisions and additions to the previous edition, although the sections mainly affected are of interest principally to mechanical and hydraulic engineers. The section on Concrete and Reinforced Concrete contains nearly 60 pages giving, briefly, information on the properties and proportioning of concrete, the design and construction of reinforced concrete buildings and bridges, and the use of prestressed concrete. Sections on the theory of structures, foundations and earthworks, mortars, and cements are of interest to structural engineers, who will also find useful information in the legal notes.

"Technical Terms used in Road Building." English-German and German-English. French-German and German-French. (Cologne: Bundesausbalt für Strassenbau. 80 pages each. Price 2.50 DM. each.)

THESE dictionaries may be useful to students of publications on roads, but some care is needed in their use as they appear to have been done in a slovenly manner. A glance at one of them shows that some of the English words are spelt wrongly, which is inexcusable in a dictionary, and German translations are given of such queer terms as compression reach, contract procision, cultivable road, enrockment, fore-arch, founderous road, maronage, strewable quantity, cubik, quater, and radix, to mention a few only of the allegedly English terms that do not exist in the English language. The work has apparently been produced by a German with insufficient knowledge of the English language. A work of reference in the nature of a dictionary should be absolutely dependable; if it is not so it should not be published.

Graphical Determination of the Grading of Combined Aggregates.

MR. F. S. FULTON, Director of the Concrete Association of South Africa, writes as follows.

The grading of any mixture of two or three aggregates of known grading can be found graphically by the following constructions.

Mixtures of Two Aggregates.

The two vertical axes and the top horizontal axis of the diagram (*Fig.* 1) are marked with percentage scales. The grading of one aggregate (in this example a pit sand) is shown on the left-hand vertical axis by marking the sieve sizes against the percentage of material passing that particular sieve, and the grading of the other aggregate (a crusher sand) is marked in the same way on the opposite axis. Straight lines are drawn to join corresponding sieve sizes, or sieve numbers, on the two axes. Any vertical line drawn between the two axes represents a mixture of the two aggregates in the proportions shown by its intersection with the top horizontal axis, and the ordinates of its points of intersection with the sloping lines joining equal sieve sizes indicate the amount of material in the mixed aggregate passing those sieve sizes.

Table I shows the grading of the two sands and the mixture, represented by the dotted line in *Fig.* 1, of 67 per cent. of pit sand and 33 per cent. of crusher sand.

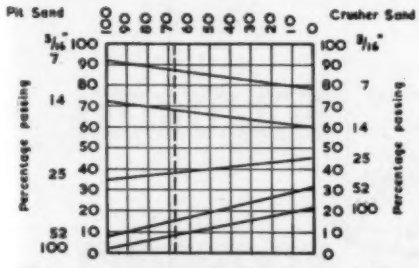

Fig. 1.—Combined Grading of Two Aggregates.

TABLE I.—GRADING OF MIXTURE OF TWO AGGREGATES.

Sieve size	Percentage passing		
	Pit sand A	Crusher sand B	Mixture 67%A : 33%B
$\frac{3}{16}$ in.	100	100	100·0
No. 7	92	79	87·5
,, 14	72	60	68·0
,, 25	35	45	38·5
,, 52	8	31	15·5
,, 100	2	21	8·5

This method is useful in obtaining the best gradings because the effect of a change in the proportions of the aggregates on the amount of material in any size-fraction can be seen.

Mixtures of Three Aggregates.

The grading of mixtures of three aggregates can be found in a similar way as

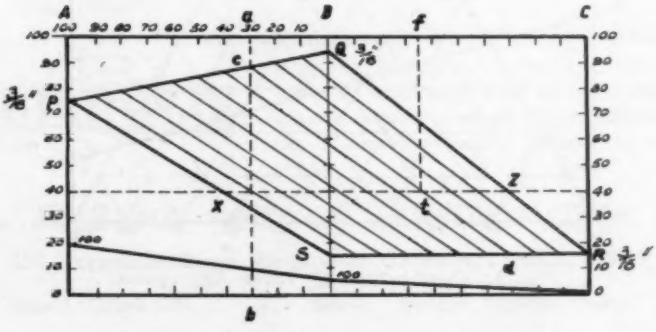

Fig. 2.—Combination of Three Aggregates.

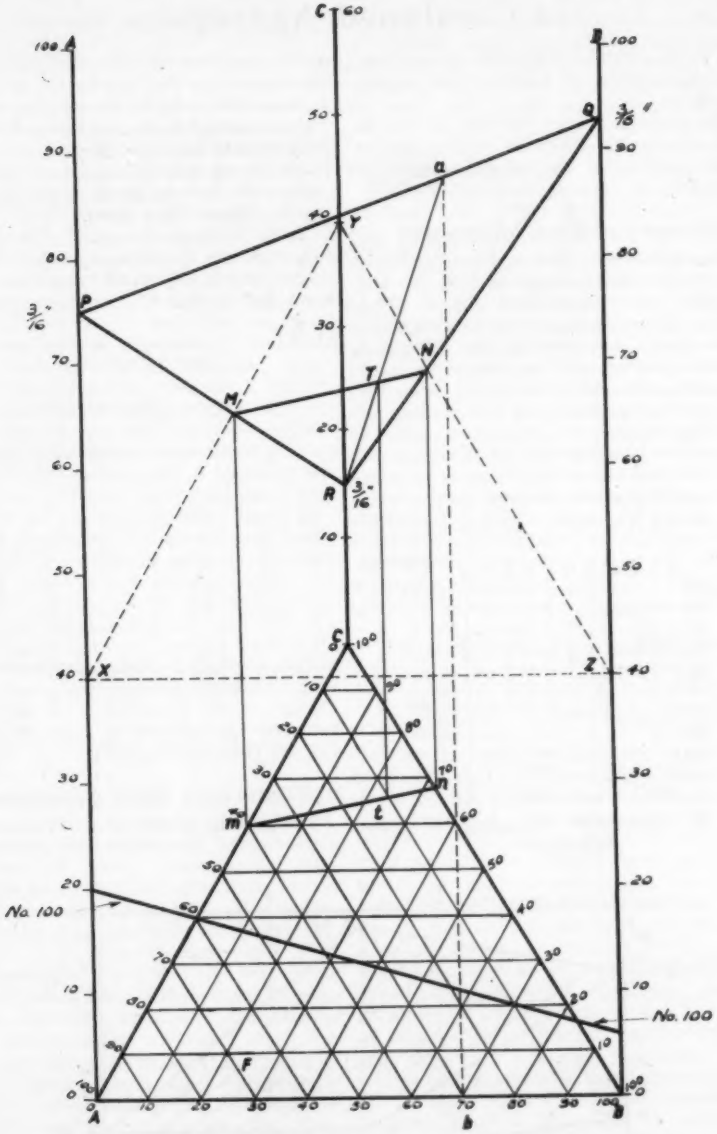

Fig. 3.—Combination of Three Aggregates.

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shown in *Fig.* 2. This method can be used for combinations of three sands, or for mixtures combining two sands with one coarse aggregate.

For example, a crusher sand *A* is to be combined with a pit sand *B* and the combined sand is to be mixed with a coarse aggregate *C* so that the proportion of sand to total aggregate is 40 per cent. and not more than 10 per cent. of the sand will pass the No. 100 sieve. The gradings of the aggregates are shown in *Table* II.

TABLE II.—GRADING OF THREE AGGREGATES.

Sieve size	Percentage passing		
	A	*B*	*C*
⅜ in.	75	93	15
No. 100	20	6	0

A diagram with three vertical axes is drawn, and the vertical and horizontal axes are marked with percentage scales. The percentages of each material passing the ⅜-in. and No. 100 sieves are marked on the respective vertical axes, and straight lines are drawn to connect corresponding sieve sizes. A horizontal line is drawn through the percentage of coarse aggregate passing the ⅜-in. sieve to cut the central vertical axis at S, and the figure PQRS is completed. Finally, guide lines are drawn across the figure joining corresponding points of intersection of the sieve lines PQ and RS with their respective percentage ordinates. The line of intersection XZ of the 40 per cent. line with the plane PQRS represents all possible combinations of the three materials for which 40 per cent. of the total aggregate will pass the ⅜-in. sieve. For a stipulated proportion of the mixture to pass a given sieve, there is only one proportion in which two aggregates may be combined, but for three aggregates any number of combinations may be chosen to produce the same result. For example, *Fig.* 2 shows that combinations of 42 per cent. *A*, 58 per cent. *C*, and 0 per cent. *B*, or of 32 per cent. *B*, 68 per cent. *C*, and 0 per cent. *A* will both contain 40 per cent. of material smaller than ⅜ in. Between these extremes, all along line XZ, any number of other combinations may be selected. The combination required is decided by requirements of the quantity of material of any size-fraction, or other considerations such as economy and availability.

The intersection of the 10 per cent. line with the line for the No. 100 sieve represents a combination of sands *A* and *B* for which not more than 10 per cent. of the mixed sand will pass that sieve size. Drawing the ordinate ab, the required proportion is 30 per cent. *A* to 70 per cent. *B*, and 87·5 per cent. of the mixture is material smaller than ⅜ in. The line ab is now treated as a new axis for the mixed sand which must be combined with the stone *C*, the new horizontal scale starts at a and the guide line cd is the line joining the ⅜-in. sieve sizes for the stone and the blended sand. Its intersection t with the line XZ shows the proportion in which the blended sand and the stone must be combined in order that the ratio of sand to total aggregate should be 40 per cent. Drawing the ordinate tf and counting back to a along the horizontal axis, the mixture required consists of 35 per cent. of mixed sand and 65 per cent. of stone. The final proportions of the mixture are (0·35 × 30) per cent. *A*, (0·35 × 70) per cent. *B*, and 65 per cent. *C*, or 10·5 per cent. *A*, 24·5 per cent. *B*, and 65 per cent. *C*.

Checking by arithmetic, the proportion of material smaller than ⅜ in. is 10·5 × 75 + 24·5 × 93 + 65 × 15 = 40·25 per cent., which shows that the graphical method is reasonably accurate.

This method does not give a clear idea of the effect of changing the proportions of the mixture, since it depends on first combining two aggregates and then combining the blend with a third aggregate. An alternative, but rather more complex, method allows effects of changes in proportions to be seen. Its application to the previous example is shown in *Fig.* 3.

First draw an equilateral triangle ABC with the sides marked with percentage scales. A grid is formed by connecting points of equal percentages to make a proportion diagram, on which any point represents a mixture of three ingredients in a certain proportion. For example, point F represents a mixture of 10 per cent. *C*, 20 per cent. *B*, and 70 per cent. *A*. At each corner of the triangle vertical axes are drawn on which percentage scales are marked. These axes correspond to the vertical axes A, B and C of *Fig.* 2,

but the new diagram represents a three-sided prism with the equilateral triangle ABC as its base. The percentages of each material passing the ¼-in. and No. 100 sieves are then marked as before on the respective vertical axes, and lines are drawn to join corresponding sieve sizes. The figure PQR now represents an inclined plane bounded by the sides of the prism, and all points above it represent material larger than ¼ in., and all points below it represent sand. By joining points of equal percentages, in this case the 40 per cent. points, on the three vertical axes, a plane XYZ is formed parallel to the prism base. This plane intersects the ¼-in. plane PQR along a line MN, and the projection mn of this line on the base ABC represents all possible combinations of the aggregates *A*, *B*, and *C* for which the material smaller than ¼ in. forms 40 per cent. of the total aggregate.

The required combination is found by considering the limitation on the amount of sand which is to pass the No. 100 sieve.

The intersection of the 10 per cent. line with the No. 100 sieve line in the plane AABB represents the desired blend of sands, and the ordinate ab through this point shows that the required proportions are 70 per cent. *B* and 30 per cent. *A*, as before. The ordinate intersects the ¼-in. line PQ in the same plane at point a. Joining a to R in the plane PQR, aR intersects MN at T. The projection t of T on the base ABC represents the required mixture, namely 10 per cent. *A*, 24 per cent. *B* and 66 per cent. *C*. The same result could of course be achieved by joining Cb. This agrees sufficiently well for graphical methods with the result from the previous construction.

Although the same result could be obtained by trial and error using simple arithmetic, the graphical method makes it possible to see the inter-relation between proportions and sieve fractions.

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Cements for Massive Concrete Structures.

A PUBLICATION* compiled by Professor M. Roš and issued recently on behalf of the Swiss Cement, Lime and Gypsum Manufacturers' Association gives a survey of the materials and methods of construction used in the building of large dams in Europe during the past five years. The publication of this volume of 150 pages coincides with the 75th anniversary of the founding of the Association. The experience gained in Switzerland and elsewhere in the construction of massive concrete structures, and particularly in the choice of cement and the methods of placing the concrete, is of wide interest, and in the following are given the salient points discussed and the recommendations made.

In his opening remarks the author states that there is no generally satisfactory solution to the problem of concreting massive structures, from the points of view either of deciding on the most suitable cement or the method of placing the concrete. Three types of hydraulic binders based on ordinary Portland cement are considered by the author to be the most suitable now available.

(1) Ordinary Portland cement as specified in the 1953 edition of the Swiss standard but with a tricalcium silicate ($3CaO.SiO_2$) content of 50 per cent. or less, and a tricalcium aluminate ($3CaO.Al_2O_3$) content of not more than 10 per cent. The cement is ground to about 7 per cent. residue on a sieve with 4900 apertures per square centimetre and should have a surface area of 2750 sq. cm. per gramme as determined by the Blaine method.

(2) Ordinary Portland cement according to the 1953 Swiss standard but containing an air-entraining agent or a chemical with a plasticising action as well as a small tendency to entrain air.

(3) Ordinary Portland cement according to the 1953 Swiss standard with the addition of slag of known chemical composition, and ground to a surface area (Blaine) of 3500 to 4500 sq. cm. per gramme.

With these cements the compressive and tensile strengths, workability, permeability, resistance to frost and changes of temperature, and resistance to chemical attack were all satisfactory. On the other hand, no simple and effective way was found to reduce the heat of hydration of cement to a level that would prevent completely uneven temperature gradients in large masses of concrete, with the consequent development of internal stresses, shrinkage, and cracking.

TABLE I.—Heat of hydration of cements used for dams in Switzerland (in cal. per kg.). Test results obtained by heat of solution method.

TYPE OF CEMENT	AGE IN DAYS							
	7		28		90		570	
	PROCESS		PROCESS		PROCESS		PROCESS	
	DRY	WET	DRY	WET	DRY	WET	DRY	WET
Ordinary Portland	71·0	–	79·5	–	85·1	–	91·2	–
Portland : slag (70 : 30)	66·9	64·0	75·6	72·5	80·9	75·4	86·9	83·9
Portland : slag (50 : 50)	61·0	53·6	70·1	67·4	74·4	70·1	79·6	75·3
Portland : slag (30 : 70)	50·9	40·0	61·4	49·5	66·8	58·6	71·7	63·8

TABLE II.—Shrinkage of Swiss cements used in massive concrete structures. (Tests on prisms 20 cm. by 20 cm. by 60 cm. containing 390 lb. of cement per cubic yard of concrete; maximum size of aggregate 2¾ in.)

CURING TIME	7 DAYS			28 DAYS			90 DAYS		
TYPE OF CEMENT	SHRINKAGE (PER CENT.)								
	RELATIVE HUMIDITY		CLOSED CYLINDER	RELATIVE HUMIDITY		CLOSED CYLINDER	RELATIVE HUMIDITY		CLOSED CYLINDER
	50 per cent.	70 per cent.		50 per cent.	70 per cent.		50 per cent.	70 per cent.	
Ordinary Portland	0·05	0·1	0·057	0·194	0·15	0·051	0·40	0·3	0·014
Portland : slag (30 : 70)	0·1	0·1	0·057	0·287	0·26	0·008	0·49	0·4	0·134
Portland : slag (50 : 50)	0·1	0·1	–	0·3	0·28	–	0·49	0·41	–

For uniformity and reliability under practical conditions, type (1) cement is considered to be the best. This conclusion is arrived at as a result of experience for many years under a variety of conditions and applications. Concrete made with this cement can be placed at the rate of about 10,000 cu. yd. per 24 hours, and such concrete has a strength of 5000 lb. per square inch at 28 days and about 5700 lb. per square inch at 90 days. As a result of a reduction in the tricalcium silicate and tricalcium aluminate contents there is a reduction in its normal heat of hydration (see *Table* I) and shrinkage (*Table* II).

* Zemente für Grosse Talsperren, by Professor Dr. Ing. h.c. M. Roš. 1956. (Verein Schweizerischer Zement-, Kalk-, und Gipsfabrikanten. Zürich. Price 15 Swiss francs.)

Among the advantages of type (2) cement the author mentions increased plasticity and workability of concrete, considerable increase in impermeability, increase in compressive and tensile strengths, improvement in resistance to frost, a slightly lower heat of hydration and tendency to cracking (although this depends to some extent on the type of air-entraining agent used), and economy in the use of cement.

Type (3) cement was of two kinds. In one the proportion of slag was between 30 and 85 per cent., and in the other it was less than 30 per cent. None of these cements is suitable for massive structures in the proportions of 250 lb. to 500 lb. of cement per cubic yard of concrete.

As regards concrete mixtures and the method of placing the concrete in large dams, the author emphasises that the quality of the cement and aggregate should be as uniform as possible; the concrete must be well compacted to enhance its resistance to moisture, frost, and changes of temperature; the heat of hydration should be as low as possible (in some cases it may be advisable to use artificial cooling while the concrete is hardening).

Air-entraining agents ensure better dispersion of the cement, with consequent reduction in the amount of gauging water required without reduction of workability, especially when crushed sand or silica aggregate is used (the use of air-entraining agents was found also to reduce the time of construction). Laboratory tests are not always a reliable guide to the behaviour of concrete, and should always be verified by tests on the site.

Portland blast furnace cement has advantages only if the properties and the mineralogical composition of the slag are uniform.

In the author's opinion the modified standard Portland cement No. (1) will remain for a long time the best for massive concrete structures. Experience shows that the most suitable maximum size of aggregate is 4¾ in. Natural quartz (silica) is preferable, but mixtures of natural (round grained) and crushed quartz are generally used. Washed natural sand is recommended. The fine sand should be separated into sizes of 0.1 mm. to 1 mm. and 1 mm. to 3 mm.; sand finer than 0.1 mm. should not be used. The use of aggregate larger than 4¾ in. is not recommended because of the difficulty of ensuring a homogeneous structure.

Large Cooling Towers.

THREE cooling towers, which are stated to be the largest in the world, are to be built at a cost of about £750,000 at the new power station at High Marnham, Nottinghamshire, for the Central Electricity Authority. Provision has been made for two additional towers.

Each tower will be 349 ft. high from the bottom of a pond of about 266 ft. diameter at the base, and will cool a maximum of 6,000,000 gallons of water an hour (atmospheric conditions of 55 deg. F. dry bulb and 70 deg. F. humidity) from 87 deg. F. to 70 deg. F., which is circulated through a concrete pipe of 6 ft. 3 in. diameter connected to a central feeder. The cooling system is a closed circuit using river water only.

The towers will be supported on diagonal columns 18 in. diameter. The ponds will be 7 ft. deep, and each will have a capacity of 2,500,000 gallons. A new type of climbing scaffold is to be used in the construction of the towers. The work is intended to be completed by September 1960. Messrs. Freeman, Fox & Partners are the consulting engineers for the power station. The contractors for the cooling towers are the Mitchell Construction Co.

Congress on Prestressed Concrete.

THE third international congress of the Federation Internationale de la Précontrainte is to be held in Berlin from May 5 to 10, 1958. Further information may be had from the Deutscher Beton-Verein, P.O. Box 543, Wiesbaden, Germany.

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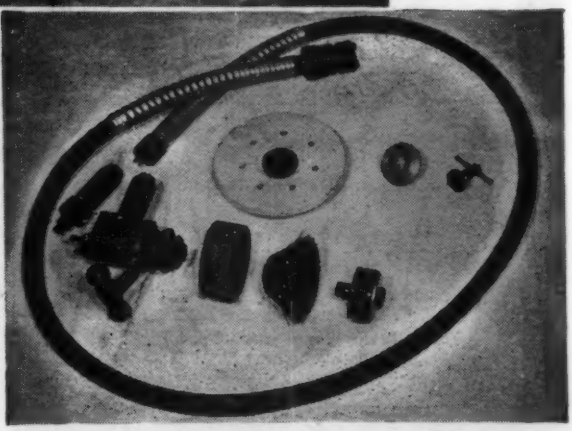

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Automatic Assembly of Reinforcement.

THE following notes and the accompanying illustrations on the use of machines for assembling reinforcement in Russia are taken from "Die Bautechnik," March 1956.

The reinforcement is generally spot welded. *Fig.* 2 shows a welded cage for a precast concrete beam. There are no bent-up bars and, instead of stirrups, vertical bars are welded to the horizontal bars.

Fig. 1 illustrates the fixing of a cage on a welding machine. The bars lie on a hand-operated movable jig which passes the points to be welded between the electrodes. After the bars on one side have been welded the jig is rotated on a turntable and the other joints are welded.

On an automatic conveyor for the assembly of reinforcement (*Fig.* 3), which is operated by three men, the following operations are carried out: Measuring and cutting of main bars; sorting of the bars and arranging them into the required positions; welding the transverse bars. The time required to produce a cage is 6 to 7½ minutes.

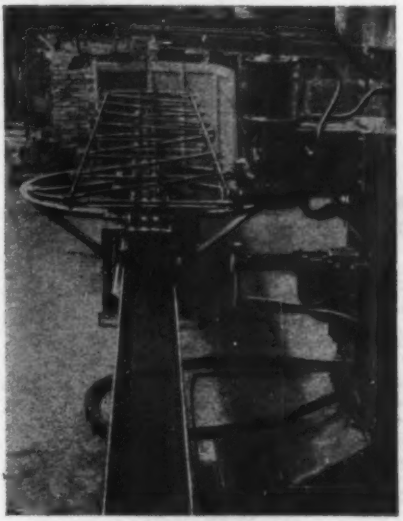

Fig. 1.—Welding Machine.

Fig. 2.—Welded Reinforcement.

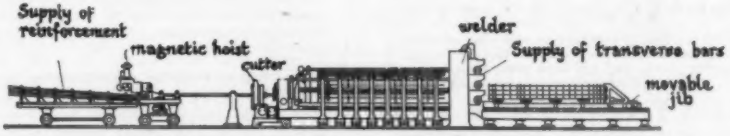

Fig. 3.—Automatic Reinforcement Welding Machine.

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A GREAT deal of useful information on the properties of wire for prestressed concrete is given in a small book issued by British Ropes, Ltd., of Doncaster, from whom copies may be had free of charge. In addition to data on the properties of high-tensile wire and the mode of testing, illustrations are given of structures and precast products in which the Company's wire has been used.

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A NEW refractory cement, known as "Secar 250," is now supplied by the Lafarge Aluminous Cement Co. Ltd. This cement withstands temperatures up to 3300 deg. Fahr., compared with the melting-point of 2460 deg. F. of high-alumina cement. A booklet describing the properties and uses of this cement may be had from the makers at 73 Brook Street, London, W.1.

New Bridge at Inverness.

THE Inverness Town Council is to construct a reinforced concrete bridge, at a cost of £400,000, to replace the suspension bridge over the river Ness. The bridge will have an overall width of 62 ft., a divided carriageway 44 ft. wide, and two footpaths each 9 ft. wide. The consulting engineers are Sir Murdoch MacDonald & Partners.

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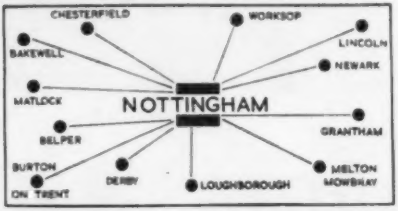

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Applicants must hold a degree in Engineering, or its equivalent. The course will include post-graduate lectures, design, drawing and laboratory work.

Applications, giving full details of qualifications and experience, must be received by the Registrar, The University, Leeds, 2, not later than 1st June, 1957.

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NOTICE IS HEREBY GIVEN that the election to Bursaries in Concrete Technology, tenable as from October, 1957, will take place in June, 1957.

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Bursaries are of the value of £460, out of which the College Tuition Fee of £60 has to be paid; the amount may be increased to a maximum of £760, depending on the qualifications and length of industrial experience. The Bursary is for the normal College session extending from October until the following June.

The course will be postgraduate, and Bursars who successfully complete the course will be eligible for the award of the Diploma of the Imperial College (D.I.C.).

Applications must be received on or before 1st June, 1957, by the DEPUTY REGISTRAR, CITY AND GUILDS COLLEGE, Exhibition Road, London, S.W.7, who will, on written request, send full information and application forms.

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SITUATIONS VACANT. Consulting structural engineers in new modern offices conveniently situated in Central Liverpool have vacancies for reinforced concrete and structural steel designer-detailers, in the salary range of £700–£1100 per annum. Special opportunities exist for those interested in nuclear energy projects, and appointments in this field carry additional increments in salary. Positions offered are permanent with excellent prospects. Staff bonus and superannuation schemes are in operation. Apply in confidence, giving age and full details of experience, to BOX 4321, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete designer-draughtsman for work in Sunbury or London office. Five-days' week. Salary £800–£1000 according to experience. Apply, stating age and details of experience, to J. H. COOMBS & PARTNERS, Consulting Engineers, Thames Corner, Sunbury-on-Thames, Middx.

SITUATIONS VACANT. R. L. BOURQUI & PARTNERS, Thermix House, Kingston-on-Thames, require detailers immediately for reinforced concrete work. Only first-class draughtsmen with good printing and linework considered. Knowledge of design an advantage, but ability to detail from calculations essential. Should be able to construct general arrangement drawings from sketches of industrial structures, framed buildings, etc. Above-average salaries offered to competent and conscientious men. Full holidays this year. Telephone for appointment to Elmbridge 0439 (office hours) or Elmbridge 2654 (evenings).

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LONDON COUNTY COUNCIL

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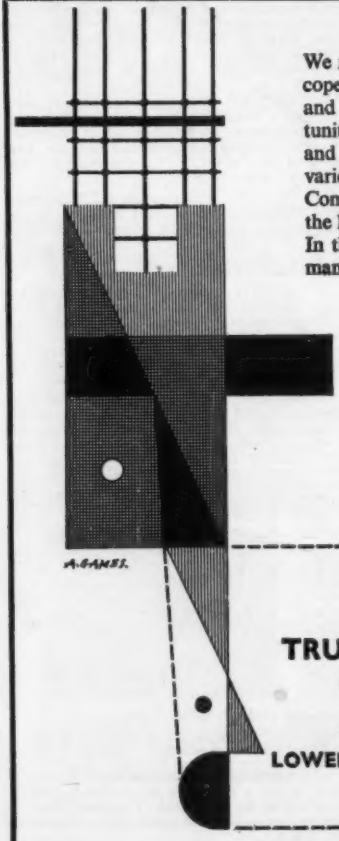

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